Appendix F: Geotechnical Investigation

GEOTECHNICAL INVESTIGATION
MARINA PARK PROJECT
NEWPORT BEACH, CALIFORNIA

Prepared for
CITY OF NEWPORT BEACH
Newport Beach, California

Prepared by
TERRACOSTA CONSULTING GROUP, INC.
San Diego, California

Project No. 2573
August 7, 2008

TerraCosta Consulting Group

TerraCosta Consulting Group

Geotechnical Engineering
Coastal Engineering
Maritime Engineering

Project No. 2573
August 7, 2008

Mr. Mark S. Reader, P.E.
Public Works Department
CITY OF NEWPORT BEACH
3300 Newport Boulevard
Newport Beach, California 92663

GEOTECHNICAL INVESTIGATION
MARINA PARK PROJECT
NEWPORT BEACH, CALIFORNIA

Dear Mr. Reader:

In accordance with your request, our Proposal No. 08018 dated March 3, 2008, and our Professional Services Agreement dated March 25, 2008, TerraCosta Consulting Group, Inc. (TCG) has completed a geotechnical investigation in support of the proposed Marina Park Development project, located on Newport Harbor between 15th and 19th Streets, and north of West Balboa Boulevard, in the City of Newport Beach, California.

The accompanying report presents the results of our review of available reports, plans, literature, our field investigation, and our conclusions and recommendations pertaining to the geotechnical aspects of the proposed site development.

We appreciate the opportunity to be of service and trust this information meets your needs. If you have any questions or require additional information, please give us a call.

Very truly yours,

TERRACOSTA CONSULTING GROUP, INC.

David B. Nevius

David B. Nevius, Project Engineer
R.C.E. 65015, R.G.E. 2789

Braven R. Smillie

Braven R. Smillie, Principal Geologist
R.G. 402, C.E.G. 207

Walter F. Crampton, Principal Engineer
R.C.E. 23792, R.G.E. 245

WFC/DBN/BRS/jg
Attachments

(6) Addressee

4455 Murphy Canyon Road, Suite 100 ▲ San Diego, California 92123-4379 ▲ (858) 573-6900 *voice* ▲ (858) 573-8900 *fax*
2601 Ocean Park Boulevard, Suite 110 ▲ Santa Monica, California 90405 ▲ (310) 399-8190 *voice* ▲ (310) 399-8195 *fax*
www.terracosta.com

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
MARINA PARK PROJECT
NEWPORT BEACH, CALIFORNIA

1 INTRODUCTION AND PROJECT DESCRIPTION

TerraCosta Consulting Group, Inc. (TCG) has performed a geotechnical investigation, and geologic and engineering analyses for development of the Marina Park project, located on Newport Harbor between 15th and 19th Streets, and north of West Balboa Boulevard, in the City of Newport Beach, California (please refer to Figure 1, Vicinity Map/Boring Location Map).

This report presents the results of our field investigation, laboratory testing, and analyses, and provides geotechnical engineering recommendations for grading and construction of the proposed improvements.

We understand that the principal structural elements of the project are:

- A 10,190-square-foot, two-story, steel-framed community center building;
- An 11,000-square-foot, two-story, steel-framed sailing center building (potentially including a 60± foot tall steel moment-frame tower);
- Two small single-story restroom structures (one of which is located approximately a block away from the site on a separate property);
- An 800-square-foot, single-story marine services building;
- Ancillary concrete flatwork and paved parking areas designed to support all of the above structures; and
- Offshore facilities, including 28 floating-dock boat slips, flexi-float support docks, approach piers, a groin-wall, and bulkheads located in an area that must be dredged to accommodate the new facilities.

The overall project layout is shown on the Architectural Master Plan, Figure 2.

2 PURPOSE AND SCOPE OF INVESTIGATION

The purpose of this investigation is to provide information to assist the City and its consultants in evaluating the site (both onshore and offshore) for project design. In particular, our investigation is designed to address the following geotechnical issues.

2.1 Onshore Facilities

- The geologic/geotechnical setting of the site;
- Potential geologic hazards, such as faulting and seismicity;
- General engineering characteristics of the identified soil and geologic units, including on-site allowable soil-bearing and earth pressure values;
- Settlement estimates;
- The depth to groundwater;
- Building foundation and flatwork recommendations;
- Building setbacks for any foundation impacts from adjacent and nearby structures, if applicable;
- Grading and earthwork recommendations; and
- Soil corrosion potential.

2.2 Offshore Facilities (Proposed ADA Approach Piers, Floating Docks, Groin-Wall, and Bulkhead Walls)

- Geotechnical recommendations for dredging;
- Geotechnical design input for the proposed groin-wall;
- Recommendations for the lateral support of the dock-area bulkheads, including both earth-anchor and tieback/deadman approaches;
- Geotechnical recommendations for approach pier foundations; and
- Depth and load/deflection criteria for use in guide pile design.

To further our understanding of the Marina Park Development, and to establish working relationships with the City's team members, we attended a project kick-off meeting on April 4, 2008, and subsequently exchanged technical information with the design team.

3 FIELD AND LABORATORY INVESTIGATION

3.1 Field Investigation

Our field investigation, performed May 16, 2008, included a geotechnical reconnaissance of the site and surrounding area; drilling, sampling, and logging two 8-inch-diameter exploratory test borings to a depth of 31.5 feet; and performing twelve continuous cone penetration test (CPT) soundings to depths ranging from 30 feet to 50 feet. The approximate locations of our test borings and CPT soundings are shown on the Boring Location Map (Figure 1). Samples were obtained from the test borings using both a 2-inch O.D. Standard Penetration Test Sampler (SPT) and a 3-inch O.D. "California Sampler." The samplers were advanced by driving them into the soil ahead of the auger using a 140-pound hammer falling 30 inches. Samples obtained from the borings were sealed in the field to preserve in-situ moisture, and transported to the laboratory for additional inspection and testing. The drilling operations were observed, and the borings logged and classified, by a geologist from our firm.

Field logs of the materials encountered in the test borings were prepared based on visual examination of the materials, and on the action of the drilling and sampling equipment. The descriptions on the logs are based on our field observations, sample inspection, and laboratory test results. A Key to Excavation Logs is presented in Appendix A as Figure A-1, and the final logs of the test borings are presented as Figures A-2 and A-3.

CPT soundings were performed at the locations of proposed structures in order to obtain continuous profiles of the underlying foundation soils, in correlation with data from the test borings. Results of the CPT soundings are also included in Appendix A.

3.2 Laboratory Testing

Representative soil samples obtained during our field exploration program were tested in the laboratory to verify field classifications and to provide data for geotechnical input to the design of project structures. The results of our laboratory tests are presented in Appendix B.

4 GENERAL SITE CONDITIONS

4.1 Geologic Setting

The project site is situated on the landward side of a naturally-formed coastal bar (or "barrier") of the type formed by a transgressive sea and littoral currents at the seaward edge of a stream delta or lagoon. The Newport Bay coastal estuary was originally formed as the lower reach of the Santa Ana River. However, in 1915, because of severe silting that resulted from flooding of the Santa Ana River (and also the construction of a man-made channel), the Santa Ana River was structurally realigned and the bay is currently fed only by San Diego Creek, which drains a comparatively small area.

4.2 Site Topography and Bathymetry

Elevations across the site range from approximately 7.8 feet (NAVD 88) along West Balboa Boulevard, ascending to almost +10 feet near the central backbone of the parcel, then back down to about +5 feet at the U.S. bulkhead line generally along the existing shoreline. From the U.S. bulkhead line, the nearshore bay floor slope descends at an inclination of approximately 10:1, down to approximate elevation -10 to -12 feet along the channel limit line.

4.3 Soil and Geologic Units

The site is underlain by hydraulic fill, bay deposits, and older alluvial deposits beyond the depths of our deepest exploratory testing at 50 feet. These soil and geologic units are described below in order of increasing age.

Hydraulic Fill Soils: Our test borings indicate that the project site-area is generally underlain by from 5 to 6 feet of loose to medium dense, gray-brown, damp to wet, hydraulically-placed sands and silty sands (SP/SM), with occasional shell fragments. It is likely that these relatively "clean" granular soils were placed as the result of dredging during one or more phases of the development of Newport Harbor. SPT blow counts within these artificially placed, dry to saturated sands range from 7 to 25 blows per foot.

Bay Deposits: The hydraulic fill sands are typically underlain by a 2- to 2½-foot-thick, soft to firm, compressible sandy silt to silty clay bay mud, which is in turn underlain by relatively clean, medium dense, gray sands (SP/SM), with shells and shell fragments, characteristic of Holocene-age bay deposits below an elevation of approximately -2 to -3 feet. SPT blow counts within these clean, saturated, natural bay deposit sands range from 13 to 24 blows per foot.

Older Alluvial Deposits: Dense to very dense, red-brown to gray, coarse "clean" sands (SP-SM), generally characteristic of older fluvial/alluvial deposits, underlie the project site area at elevations ranging from approximately -20 to -26 feet. Limited blow counts within these older estuarine soils range from 37 to 38 blows per foot. However, the CPT tip resistance in these deposits typically exceeds 300 tsf, indicating a very dense sand.

4.4 Groundwater

Groundwater levels at the site can be expected to vary in response to tidal fluctuations. Groundwater highs will likely approach tidal highs in the bay, and groundwater lows may drop slightly below mean sea level. From a construction standpoint, any excavations advanced down to within the tidal zone should be expected to experience severe caving.

5 GEOLOGIC HAZARDS

5.1 Regional and Local Faulting

We did not observe indications of faulting during our field investigation at the site, and available geologic literature does not indicate that active faults have been mapped in the immediate project site area. However, our review of published and unpublished maps indicates that the site is approximately 3 km westerly of the Newport-Inglewood/Rose

Canyon fault zone (south Los Angeles Basin segment), which generally trends north/south along the easterly margin of the Newport ("Upper") Back Bay. It is generally accepted that movement along the Newport-Inglewood/Rose Canyon fault zone has created compressional forces, which caused warping and tilting of the portion of crustal block underlying this area of Orange County.

5.2 **Seismicity**

The project site is located in a moderately active seismic region of Southern California that is subject to moderate to strong shaking from nearby and distant earthquakes. Ground shaking from earthquakes on 63 major active faults could affect the site. The nearest of these, the Los Angeles Basin segment of the Newport-Inglewood/Rose Canyon Fault, is located approximately 3 km easterly of the site. According to the United States Geologic Survey (USGS) Open-File Report 2008-1128, the maximum credible earthquake for this segment of the Newport-Inglewood/Rose Canyon Fault is considered to be magnitude 7.2. During the 1933 Long Beach earthquake, a 6.4 magnitude shock was experienced offshore approximately 2.5 miles north-northeast of the site about 30 minutes prior to the shocks that devastated Long Beach.

We used both the California Geologic Survey (CGS) and the USGS Probabilistic Seismic Hazards web sites to assess the probabilistic ground motion conditions of the site. According to both the CGS and USGS, the peak ground acceleration for a 10 percent probability of exceedance in 50 years is estimated to be on the order of 0.37 to 0.41g.

5.3 **Geologic Hazards**

Potential geologic hazards that may exist at the site include landslides, fault rupture, ground shaking, liquefaction, seismic-induced settlement, lateral spreading, seiches, and tsunamis. With respect to these potential hazards, we have the following comments:

- **Landslides:** No landslides have been mapped at the site. As such, it is our opinion that the risk associated with landslides is negligible.
- **Fault Rupture:** No faults have been mapped across the site or inferred to cross the site. As such, it is our opinion that the risk associated with fault rupture is low to negligible.

- **Ground Shaking:** All sites within Southern California are susceptible to ground shaking.

- **Liquefaction:** Liquefaction is a potential hazard in any water-saturated, clean sandy soils. The loose to medium dense, near-surface hydraulic fills and bay deposits (typically above elevation -15 to -25 feet) exhibit relatively low relative densities and consist of clean (SP/SM) soils, making these materials susceptible to seismic-induced liquefaction and lateral spreading. The dense to very dense, older alluvial deposits encountered below -20 to -26 feet are not susceptible to liquefaction. Spontaneous liquefaction develops within sandy soils when they are subjected to a rapid buildup of pore pressure, such as that caused by seismic shock, and the result of this condition could be massive mobilization of the near-surface foundation soils and the failure (settlement) of site-area structural improvements. It is expected that liquefaction could be triggered at this site with a seismic acceleration of 0.20 g.

- **Seismic-Induced Settlement:** Ground settlements due to seismic activity results from a densification of soils due to ground vibration, as well as by reconsolidation of liquefied soils. For the facilities under consideration for this study, we anticipate that the majority of the seismic ground settlements will be associated with potential liquefaction of the upper 20± feet of the hydraulic fills and bay deposits. We estimate that if these soils were to liquefy, the amount of total induced settlement could be on the order of 1 to 4 inches.

- **Seiches:** As the site is located within the Newport Bay, it is our opinion that the risks associated with seiches are moderate to high.

- **Tsunamis:** As the site lies on the coast, it is our opinion that the risk associated with tsunamis is the same as all projects located along the shoreline of the City of Newport Beach. Studies performed by Legg, Borrero, and Synolakis (2004) suggest that this area of the coastline may be affected by both earthquake- and subaqueous landslide-generated tsunamis with wave heights of 2+ meters and wave runup of 4+ meters. As such, the site may be affected by a tsunami under certain critical conditions. As we understand, the City of Newport Beach already has a tsunami contingency plan and evacuation routes in place.

6 CONSIDERATIONS FOR LANDSIDE IMPROVEMENTS

6.1 Site Preparation

It is recommended that the entire site be scarified to a minimum depth of 12 inches, watered, and properly recompacted to a minimum of 95 percent relative compaction, in accordance with ASTM Test Designation D 1557. Any loose zones encountered during compaction of the final subgrade should be overexcavated and properly recompacted to 95 percent in order to provide the recommended subgrade density. We would recommend that the deep foundations for the Sailing Center and Community Center, whether driven piles or stone columns, be completed prior to the completion of subgrade preparation.

We recommend that the existing hydraulic fill sands be compacted by a combination of flooding and vibration using a vibratory roller, compactor, or heavy track equipment.

All site preparation and grading should be performed under the observation of the geotechnical engineer and in accordance with Section 300, "Earthwork," of the Standard Specifications for Public Works Construction ("Greenbook").

6.2 Foundation Design

From a geotechnical standpoint, the near-surface hydraulic fill sands are relatively competent in nature and suitable for supporting relatively lightly loaded foundation elements assuming sufficient confinement of the near-surface soils. However, given the potential for liquefaction and liquefaction-induced settlements that could be on the order of 1 to 4 inches, we recommend using a deep foundation system, or soil improvement with a mat foundation for the Sailing Center and Community Center. We recommend that mat foundations be used for smaller proposed buildings, including restroom facilities.

6.2.1 *Mat Foundations for Restroom Facilities and Other Small Buildings*

We recommend that all mat foundations be designed by a registered civil or structural engineer experienced in mat foundation design. We recommend a subgrade modulus of 100 pci, which has been adjusted for foundation size. We recommend that maximum allowable contact stresses be limited to 1,000 psf. This value should not be increased for any transient loads, including seismic and wind loads.

To provide resistance for design lateral loads, we recommend that an allowable friction coefficient of 0.45 be used between the concrete mat foundation and the underlying recompacted sandy subgrade soils. If, for some reason, additional lateral resistance is required, interior shear keys can be added when located a minimum of three times the depth of the shear key in from the perimeter edge of the mat foundation. Passive pressures, if used, should be limited to an equivalent fluid pressure of 300 pcf.

6.2.2 *Deep Foundations for Sailing Center and Community Center*

Due to the potential for significant settlement due to liquefaction, we recommend that the Sailing Center and Community Center buildings be supported on either driven piles, or on structural mats, the latter of which should be supported by improved soil. We recommend stone columns be used to densify the underlying soil if mats are the chosen foundations for the Sailing Center and Community Center. Both of these foundation alternatives are discussed in the following paragraphs.

<u>Pile Foundations</u>

In order to avoid undesirable liquefaction-induced settlements, we recommend that consideration be given to supporting all settlement-sensitive habitable structures on pile foundations deriving their support from the dense alluvial sands encountered below elevation -26 feet. As indicated in Section 5.3, potentially liquefiable sands overlie these dense sands and, under the design earthquake event, may locally liquefy down to a maximum elevation of about -26 feet, resulting in potential downdrag forces imposed on the upper portions of foundation piles. We currently anticipate maximum liquefaction-induced downdrag loads applied to 12-inch square pre-stressed concrete piles approaching 50 kips and recommend that all pile foundations be designed to accommodate this additional seismically induced axial downdrag load.

We recommend that 12-inch square pre-stressed concrete piles be designed for a minimum of 10 feet of embedment into the dense to very dense alluvial sands corresponding to a minimum design tip elevation of -35 feet. At this depth, the allowable bearing capacity of these soils will exceed the pile's maximum design allowable capacity of 105 tons (80 tons when subtracting out downdrag forces).

We anticipate that the dense alluvial sands will require limited pre-jetting to achieve design tip elevation and pre-jetting shall be allowed down to elevation -30 feet. However, in all instances, actual pile capacities and tip elevations shall be verified in the field utilizing a suitable pile driving formula, such as the Engineering News Record (ENR) formula.

We recommend that our firm observe the driving of all piles. Continuous records of pile driving operations should be kept and any field changes reviewed with the structural engineer. Typical guide specifications for pile driving are attached in Appendix C, and may be used as an aid in preparation of job specifications.

Stone Columns with Mat Foundations

As an alternative to conventional deep foundations, in-situ ground improvement may also be performed to densify the near-surface liquefiable soils and to improve pore pressure dissipation resulting from seismic shaking. We consider stone columns to be a viable alternative to mitigating the potential for seismically induced liquefaction and the associated ground settlements that should be expected during the design seismic event. Thirty to 36-inch-diameter stone columns placed in a typical 7-foot triangular pattern, extending to a depth of approximately 30 feet, should provide sufficient increased soil stiffness to mitigate the potential for seismically induced liquefaction and ground settlements. This in-situ densification occurs by advancing a large electric or hydraulic vibrator to the desired depth with use of water or air-jetting to assist penetration to the design depth. After penetration, the vibrator is partially withdrawn and the hole created by the vibrator filled with a charge of stone. The vibrator is again lowered into the stone, displacing the stone both radially and downward into the surrounding soil, thereby causing displacement of the soil over and above that created by the initial penetration of the vibrator. In this way, a compact column of stone interlocked with the surrounding ground is built up to the ground surface.

As indicted in Section 6.2.1, we recommend that foundations for the proposed marina buildings, if supported on stone columns, be supported by a structural concrete mat foundation, which in turn would be supported by the stone column densified subgrade soils.

6.3 Seismic Design Parameters per CBC

The California Building Code (CBC) requires a site-specific seismic response analysis for any site that is considered liquefiable. However, based on ASCE Standard ASCE/SEI 7-05, if the proposed structures have a fundamental period of vibration equal to or less than 0.5 seconds, site-specific analysis is not required and response spectra can be determined using the equivalent site class for non-liquefiable soil. In this particular case, we recommend using the Site Class D characterization for stiff soil. For this site class, we recommend using spectral accelerations of 1.252 and 0.711 for periods of 0.2 and 1.0 seconds, respectively.

6.4 Concrete Flatwork and Walkways

We recommend that areas to receive concrete flatwork and walkways be prepared in general accordance with Section 301-1 of the Greenbook Specifications. We recommend that subgrade soils be scarified to a minimum depth of 6 inches, and compacted to a minimum relative compaction of 95 percent. Additional subgrade preparation may be necessary in those areas where flatwork and walkways may be subject to vehicle loading and should be evaluated on a case-by-case basis.

6.5 Soil Corrosivity

The results of corrosivity testing of the near-surface soils indicate a soil pH of 7.0 and 40 years to perforation for a 16 gauge metal culvert. Test results are included in Appendix B.

7 CONSIDERATIONS FOR MARINA IMPROVEMENTS

7.1 Sheet-Pile Bulkheads

It is our understanding that the subject sheet-pile walls will be pre-stressed, pre-cast, concrete panels and that those panels will be installed in a sequence as generally shown on Figure 3. At the contractor’s option, we would anticipate that the sheet-pile bulkheads would be installed in a partially excavated trench and then jetted to near grade. Jetting may be permitted down to within 1 foot of design tip elevation, and then driven the last foot. Concrete sheets should use tongue-and-groove connections and should have jet tubes cast into the pile. The tongue-and-groove connection should be cast in such a way to allow installation of a 1½-inch-diameter pipe (after driving) into the oversized groove. A high-pressure water jet should be used to initially flush out any debris from within the joint. Each joint should then be pressure grouted to protect against possible loss of the soil backfill out through joints.

As shown on Figure 3, we recommend installation of the Sailing Center foundations prior to installation of the interior marina bulkhead anchors to avoid potential conflicts between the tiebacks and piles or stone columns.

We have used Shoring Suite Version 8, by CivilTech, Inc. for design of the bulkhead walls. Based on the results of our CPT data and borings, we have selected an active earth pressure coefficient of 0.31, and a passive earth pressure coefficient of 3.2, reduced to 2.25, to ensure a factor of safety of 1.5 with regard to passive toe failure. We examined the shore-parallel Sailing Center bulkhead (+9 elevation, plan datum) with and without seismic loading, as well as the interior marina bulkhead walls (+10 elevation, plan datum) with H20 vehicle loading adjacent to the wall edge without seismic loading, and with seismic loading (without the H20 surcharge). We have also assumed a 4-foot tidal lag in front of the bulkhead wall. We have neglected the presence of the sloping passive toe in front of the bulkhead walls, as these sloping toes can be partially or completely scoured out as the result of boats backing into or out of their docks. Summary calculations are provided in Appendix D.

Our analyses indicate that the critical design case for both the Sailing Center bulkhead wall and the interior marina bulkhead walls is the seismic loading condition under a design seismic acceleration of 0.20 g. For this condition, we have also increased the design acceleration by 50 percent to take into consideration the lack of deformation exhibited by rigid structures (Xanthakos, 1995).

As indicated in Sections 5.2 and 5.3, the design seismic event has a peak ground acceleration with a 10 percent probability of exceedance in 50 years estimated to be on the order of 0.37 to 0.41 g. Moreover, for the site conditions, localized liquefaction is anticipated with site accelerations exceeding 0.2 g, with massive liquefaction and lateral spreading affecting the upper 20± feet with site accelerations approaching 0.4 g. Under these conditions, the bonded zone of the tiebacks would yield, and the liquefied bulkhead backfill would then overload and fail the now-cantilevered 22-foot-high bulkhead. As the bulkhead is not a habitable structure, to our knowledge, there is no code mandate to design for the 0.4 g seismic event. However, if desired, the bulkhead could be designed to resist the maximum seismic event by densifying the liquefiable bulkhead backfill materials, as well as the bonded zone for the tieback anchors. This liquefaction mitigation can be achieved through the use of stone columns, treating the zone extending roughly 70 feet back from the bulkhead. If this were to be considered, however, we anticipate that it may be more economical to use deep soil mixing adjacent the back of the bulkhead, which should be able to mitigate the maximum design seismic event with a soil mixed zone possibly 30 feet in width.

As such, we recommend the following design parameters for the walls:

Sailing Center Bulkhead Wall

Top Elevation:	+9, plan datum
Minimum Embedment:	17 feet
Minimum Tip Elevation:	-29 feet, plan datum
Maximum Design Moment:	84 kip-ft
Required Top-of-Wall Lateral Restraint:	9.4 kips/lineal foot

Interior Marina Bulkhead Walls

Top Elevation:	+10, plan datum
Minimum Embedment:	18 feet
Minimum Tip Elevation:	-30 feet, plan datum
Maximum Design Moment:	96 kip-ft
Required Top-of-Wall Lateral Restraint:	10.3 kips/lineal foot

7.1.1 *Tieback Anchors*

We understand that deadman anchors would attach to the bulkhead within the pile cap at about elevation +9 feet (+8 feet for the Sailing Center bulkhead wall). Assuming conventional deadman anchors were used, these anchors would extend a minimum of 7 feet below grade and run continuously behind the bulkhead. Since deadman anchors cannot encroach onto the adjacent easterly parcel, and 7-foot-deep continuous deadman anchors will likely pose significant construction difficulties, we understand that it ha been agreed to use post-grouted soil anchors to restrain all site bulkheads.

Post-grouted soil anchors on tiebacks offer several significant advantages in that effective corrosion protection is assured, convenient preloading is possible, and construction conflicts with the Sailing Center deep foundations are minimized.

In this regard, we anticipate that tieback anchors would be installed on 8 to 10 foot centers. For these conditions, we recommend a minimum unbonded length of 40 feet, and a minimum bonded length of 30 feet. As indicated, we also recommend that the tieback anchors be installed at an inclination of 4 to 1 (horizontal to vertical), resulting in the tieback depth at the easterly edge of the Sailing Center building near elevation +1.5 foot.

We recommend that tiebacks be installed with the use of a casing drill, such as a Klemm, which enables advancing a cased hole to the full design embedment depth. The anchor would then be inserted into the cased hole, grouted, and then the casing removed, enabling the straightforward installation of tieback anchors in clean sands that would otherwise cave into any drilled hole.

We recommend the use of DYWIDAG Systems International (DSI) anchors, with Type C double-corrosion protection. DSI product literature is provided in Appendix E.

7.2 **Guide Pile Recommendations**

As we understand, guide piles for the proposed marina docks will utilize round pre-stressed concrete piles designed to accommodate maximum lateral design loads on the order of 2 to 4 kips. The outer shore-parallel 200-foot-long public side tie visitor dock will also be restrained by round guide piles. We also understand that this dock may incorporate a wave attenuation structure, which may ultimately result in lateral design loads on the order of 8 to 12 kips.

In order to evaluate the structural requirements and load deformation characteristics of the proposed concrete guide piles, we have used the elastic theory approach developed by Matlock and Reese (1962). A condensed version of this approach is outlined in the NAVFAC Design Manual DM 7.02, Chapter 5, Section 7. A copy of this design section is included with our calculation package (Appendix D). We have also used a coefficient of variation of soil modulus of 15 pci for the medium dense to very dense sand deposits, which extend well below the depth of interest.

Ultimate lateral load capacity was also evaluated using the approach developed by Broms (1965), which follows the general approach developed by Matlock and Reese.

We have used a roller assembly design load elevation of +10.0 feet (plan datum) and a dredge bottom elevation of -12 feet. For this loading condition, we have calculated guide pile deflections for 14-inch, 16-inch, 20-inch, and 24-inch round, prestressed concrete piles for the marina docks and the visitor dock. Figure 4 presents the load-deflection relationship for each pile size.

When using the Matlock and Reese solution, in order to minimize guide pile deflections and account for variabilities in subsurface soil conditions, we recommend a minimum embedment depth of 4T or $4(EI/f)^{1/5}$. The recommended minimum embedment depth for various pile diameters is also summarized in Figure 4. Calculations are also attached.

7.2.1 *Pre-Jetting Considerations*

Based on the subsurface data obtained from our borings, the relatively clean dense sands will require pre-jetting to reach the required design tip elevation. To maximize the lateral load capacity and minimize the deformation and response to lateral loads, jetting should be terminated approximately 2 feet from the design tip elevation, and the last 2 feet driven to aid in redensifying the soils disturbed by jetting. We would suggest the use of a minimum 50,000 foot-pound capacity pile hammer to achieve design tip elevations within the medium dense to dense alluvial soils.

The jetting of piles, and particularly if contemplated to be used to advance the piles down to design tip elevations, should be done using internal jet pipes, and jet volumes and velocities should be limited to the minimum flow needed to advance the piles. In this regard, it is important to recognize that excessive jetting will tend to enlarge the hole and significantly reduce the lateral load capacity of the soil. The proper jetting technique is to use a low-volume, low-pressure flow of water through the internal jet pipe while repeatedly lifting and dropping the pile to displace the dense sands beyond the pile tip and expel the sands up the annulus of the jetted hole without excessively disturbing the surrounding dense sands. The proper jetting technique essentially allows the lifting and repeated dropping of the pile to redensify the sand as the pile is advanced into the dense underlying sands.

7.3 **Approach Pier/Gangway Abutment Foundation Recommendations**

We understand that the interior marina will be accessed by a single ADA-compliant gangway, approximately 80-feet long. We further understand that the gangway will be attached to a square concrete abutment supported by both the southerly and easterly bulkheads, along with a single round concrete pile positioned on the outward edge of the abutment centered between the gangway hinge assembly. We recommend a minimum design pile tip elevation of -25 feet, plan datum. Jetting, if desired, may be allowed down to elevation -20 feet. We recommend an allowable axial capacity of 40 kips for a 16-inch-diameter pile. We have not considered lateral loading for this condition; however, additional design criteria can be provided, if desired.

7.4 Dredging

As we understand, other consultants have provided recommendations regarding the environmental processing of dredged materials. With regard to geotechnical considerations, it should be noted that there is a 2- to 3-foot-thick layer of clayey material near elevation +1 to +2 feet (plan datum) that may affect the dredging and disposal operations. With the exception of this relatively thin layer of soil, all of the other on-site materials consist of granular sands and would likely be suitable as beach-quality sand fill. All of the near-surface soils may be dredged using conventional dredge equipment.

7.5 Shore Perpendicular Groin-Wall

As we understand, a shore perpendicular groin-wall is also proposed to accommodate deep-water access adjacent the westerly floating dock. We would suggest that the load deformation and structural requirements for this shore-parallel bulkhead be designed utilizing the elastic theory approach developed by Matlock and Reese and described in Section 7.2. Although the same coefficient of variation of soil modulus would apply in this area, the Matlock and Reese design assumes isolated piles, with soil bridging providing an approximately threefold increase in passive resistance restraining the isolated pile. Thus, when using the NAVFAC design manual for design of the shore perpendicular groin-wall, a coefficient of variation of soil modulus of 5 pci should be used to account for the continuous shore perpendicular groin-wall.

8 LIMITATIONS

Coastal engineering and the earth sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our evaluation of the technical information gathered, partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet the current professional standards. We do not guarantee the performance of the project in any respect.

We have investigated only a small portion of the pertinent soil and geologic conditions at the subject site. The opinions and conclusions made herein were based on the assumption that the soil and geologic conditions do not deviate appreciably from those encountered during our field investigation. We recommend that a soil engineer from our office observe construction to assist in identifying soil conditions that may be significantly different from those assumed in our design. Additional recommendations may be required at that time.

REFERENCES

ASTM Standard D 1557, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort," ASTM International, West Conshohocken, PA, www.astm.org.

American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures, ASCE Standard ASCE/SEI 7-05, including Supplement No. 1 and Errata.

Broms, B.B., 1965, "Design of Laterally Loaded Piles," in Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, Vol. 91, No. SM3, May 1965, pp. 79-99.

Fuscoe Engineering, June 6, 2008, Preliminary Topographic Survey, Marina Park.

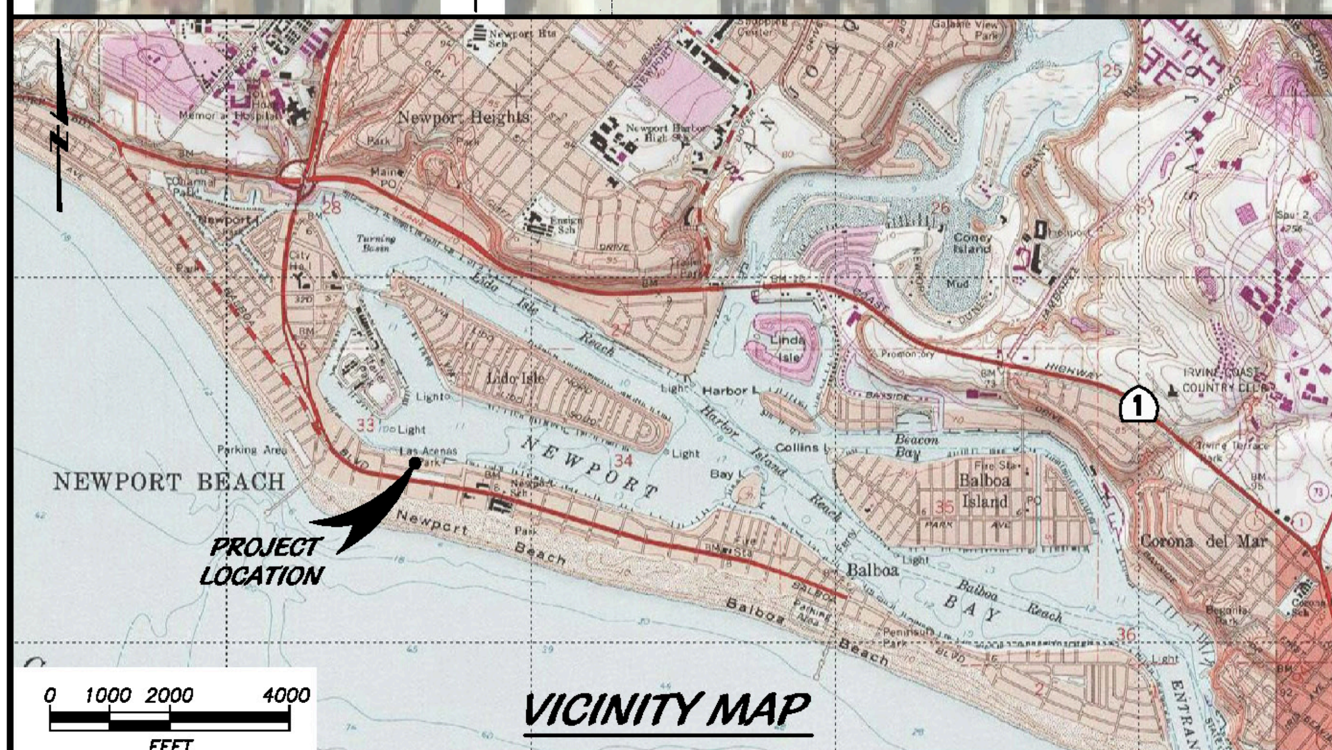
Legg, M.R., and J.C. Borrero and C.E. Synolakis, C.E., 2004, Tsunami hazards associated with the Catalina fault in southern California: Earthquake Spectra, vol. 20, p. 917-950.

Matlock, H., and L.C. Reese, 1962, "Generalized Solutions for Laterally Loaded Piles," in Transactions of the American Society of Civil Engineers, Vol. 127, Part 1, Paper No. 3370, pp. 1220-1251.

Public Works Standards, Inc., 2006, Greenbook: Standard Specifications for Public Works Construction, Building News, Inc.

U.S. Department of the Navy, Naval Facilities Engineering Command, 1986, Foundations and Earth Structures, NAVFAC DM 7.02.

Xanthakos, P.P., 1995, Bridge Substructure and Foundation Design, Prentice Hall, New Jersey.



VICINITY MAP

Base Map reproduced from City of Newport Beach Marina Park Master Plan. Base Photo taken from Google Earth image 2008.

LEGEND

B-2 APPROXIMATE BORING LOCATION

CPT-12 APPROXIMATE CPT LOCATION

APPROX. SCALE:
1"=100'

THOMAS GUIDE, ORANGE COUNTY, PAGE 918-J1

TerraCosta Consulting Group

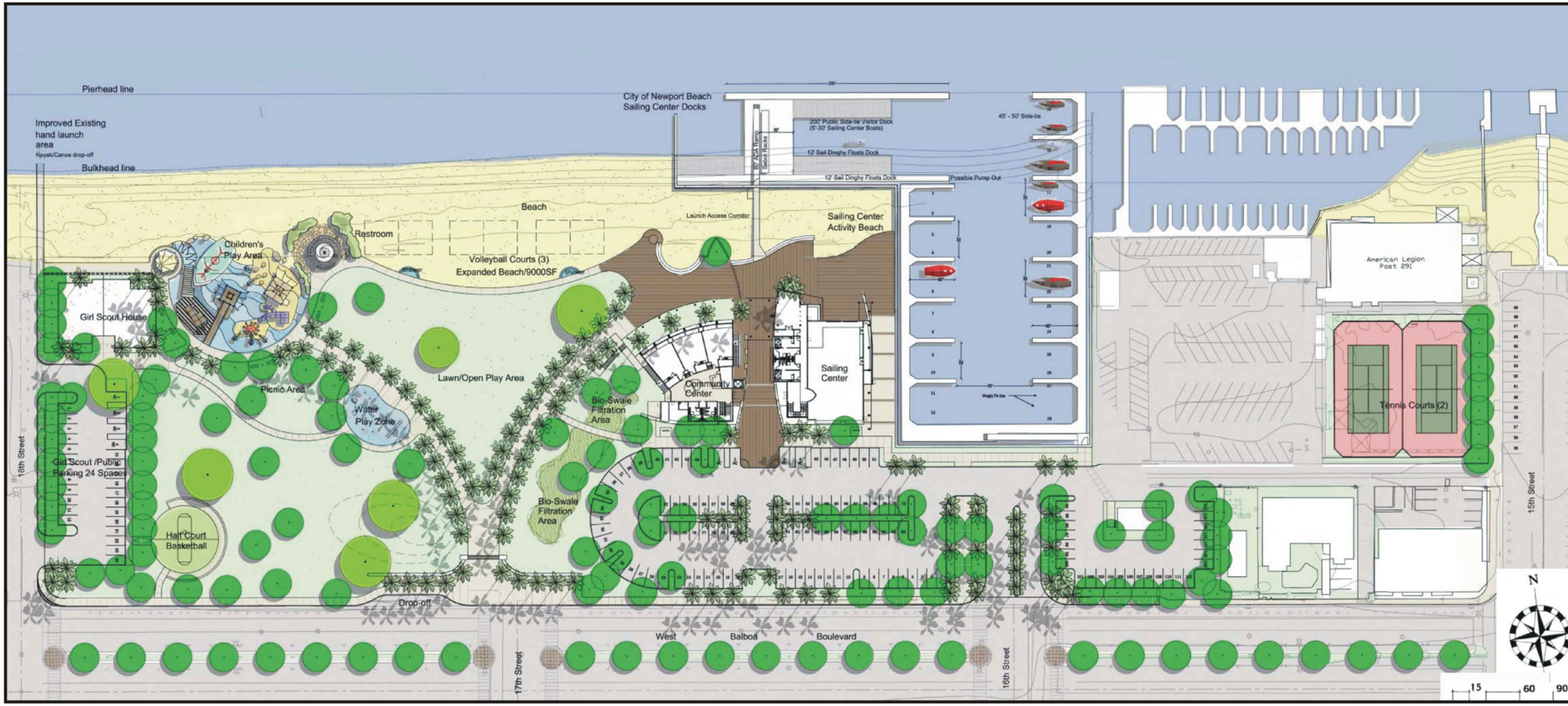
TERRACOSTA CONSULTING GROUP
ENGINEERS AND GEOLOGISTS
4455 MURPHY CANYON ROAD, SUITE 100
SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER
1

PROJECT NAME
MARINA PARK
CITY OF NEWPORT BEACH

PROJECT NUMBER
2573

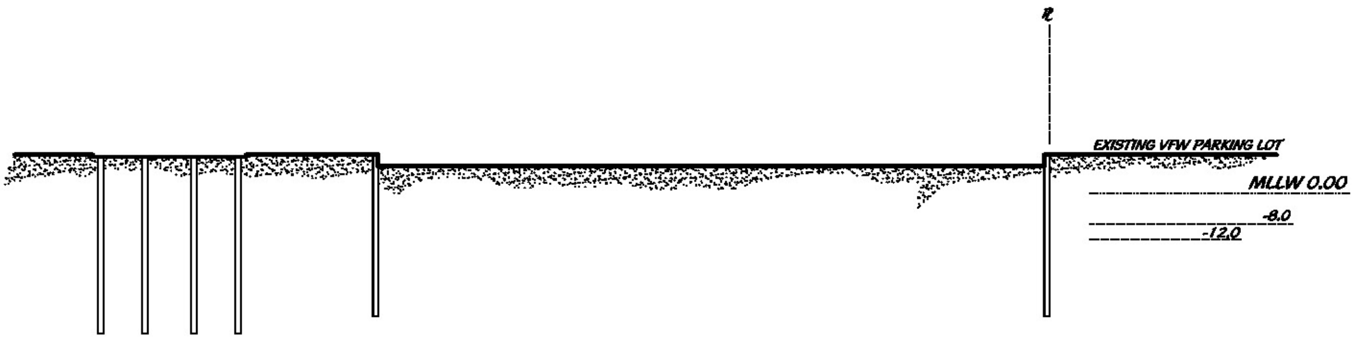
BORING LOCATION MAP



TerraCosta Consulting Group	TERRACOSTA CONSULTING GROUP ENGINEERS AND GEOLOGISTS 4455 MURPHY CANYON ROAD, SUITE 100 SAN DIEGO, CA 92123 (858) 573-6900	FIGURE NUMBER 2
	PROJECT NAME MARINA PARK	PROJECT NUMBER 2573
	ARCHITECTURAL MASTER PLAN	

1

1. INSTALL CONCRETE SHEETPILES
2. INSTALL FOUNDATION PILES FOR SAILING CENTER
3. EXCAVATE 2 TO 3 FEET OF MATERIAL BETWEEN SHEETPILES

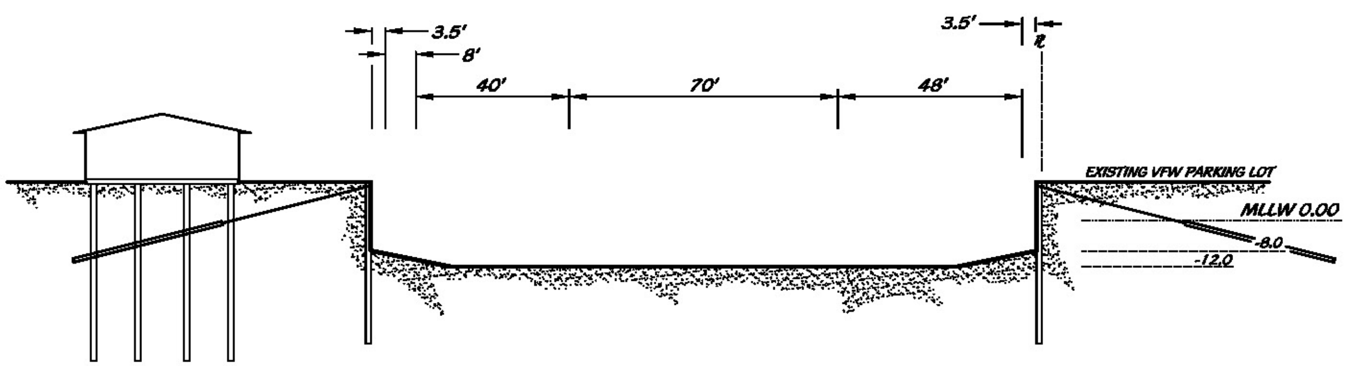


2

INSTALL TIEBACKS

3

EXCAVATE REMAINING MATERIAL BETWEEN SHEETPILES TO PLAN GRADES

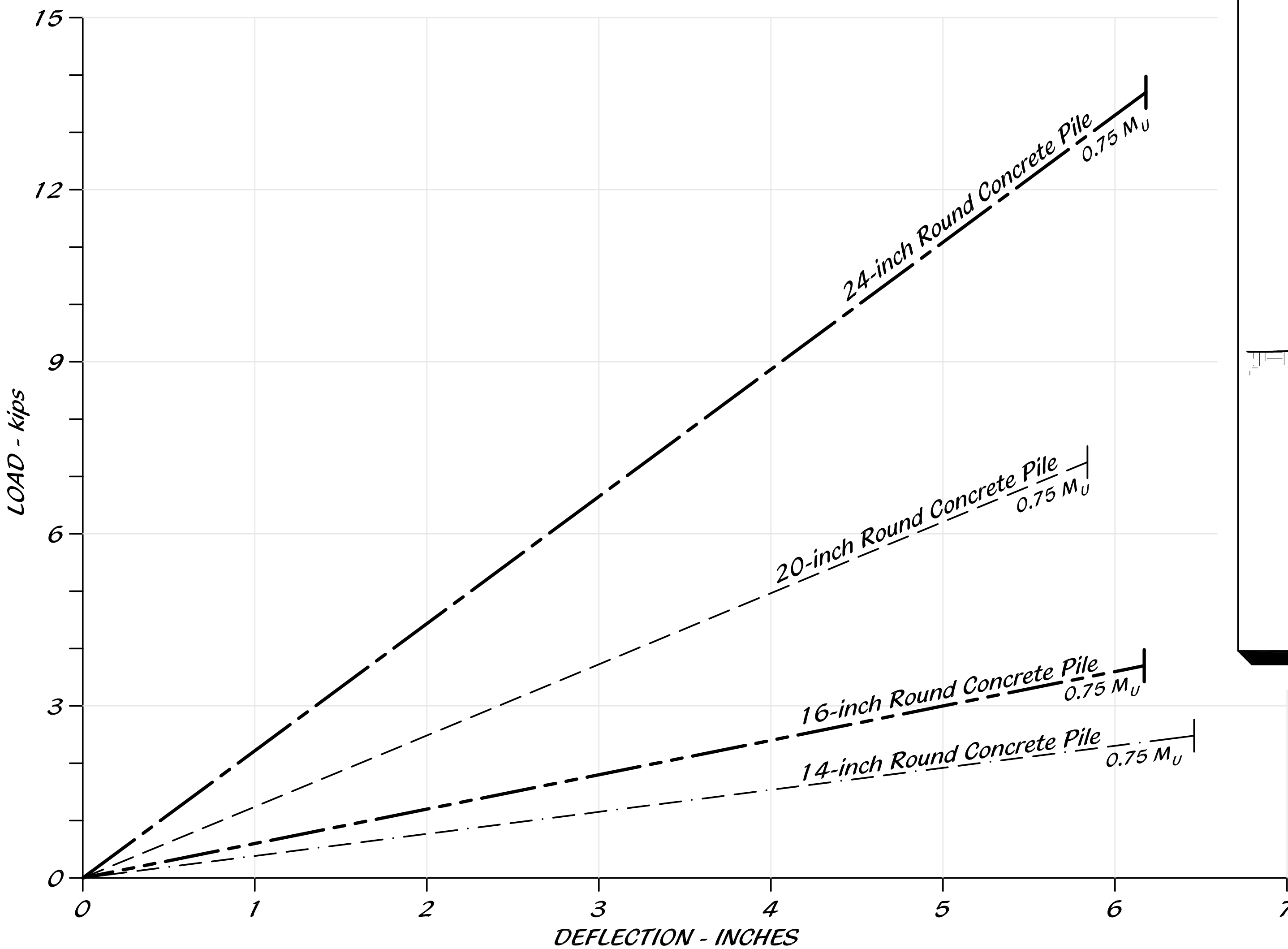


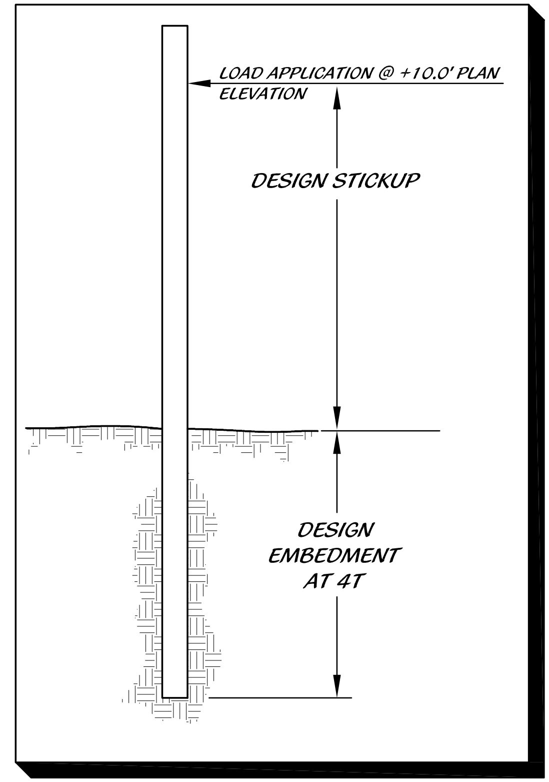
TerraCosta Consulting Group

TERRACOSTA CONSULTING GROUP
ENGINEERS AND GEOLOGISTS
4465 MURPHY CANYON ROAD, SUITE 100
SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER	PROJECT NAME	PROJECT NUMBER
3	MARINA PARK	2573

CONSTRUCTION SEQUENCE





PILE DIA. inches	*M_U, K-ft	0.75 M_U, K-ft	LATERAL LOAD @ 0.75 M_U, K	DEFLECTION, inches	MIN. DESIGN EMBEDMENT DEPTH, feet	DESIGN TIP ELEV., feet
14	77.5	58.1	2.48	6.46	18	-30
16	116.6	87.5	3.70	6.17	20	-32
20	232.5	174.4	7.25	5.84	23	-35
24	447.3	335.5	13.70	6.18	27	-39

*DATA FOR PILES MANUFACTURED BY UTILITY VAULT

TerraCosta Consulting Group

TERRACOSTA CONSULTING GROUP
ENGINEERS AND GEOLOGISTS
4455 MURPHY CANYON ROAD, SUITE 100
SAN DIEGO, CA 92123 (858) 573-6900

FIGURE NUMBER
4

PROJECT NAME
MARINA PARK

PROJECT NUMBER
2573

ROLLER DEFLECTION

APPENDIX A

LOGS OF TEST BORINGS & CPT SOUNDINGS

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
MARINA PARK	2573	LEGEND

SITE LOCATION	START	FINISH	SHEET NO.
Newport Beach, CA	5/16/2008	5/16/2008	1 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Gregg Drilling	Hollow Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl B5	8	40		▼ n/a_

SAMPLING METHOD	NOTES
140-lb hammer / 30-inch drop	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
5									
10									
		C	1						
		S	2						
15		B	3						
		PB	4						

KEY TO EXCAVATION LOGS

▼ WATER TABLE MEASURED AT TIME OF DRILLING

OTHER TESTS

CC	Confined Compression	PI	Plasticity Index
CL	Chloride Content	R	Resistivity
CS	Consolidation	RV	R-Value
DS	Direct Shear	SA	Sieve Analysis
EI	Expansion Index	HD	Hydrometer
GS	Grain Size Analysis	SF	Sulfate
LC	Laboratory Compaction	SG	Specific Gravity
pH	Hydrogen Ion	SW	Swell

PENETRATION RESISTANCE (BLOWS/ft)

Number of blows required to advance the sampler 1 foot.

California Sampler blow counts can be converted to equivalent SPT blow counts by using an end-area conversion factor of 0.67 when using a 140-pound hammer and a 30-inch drop.

SAMPLE TYPE

C ("California Sampler")- An 18-inch-long, 2-1/2-inch I.D., 3-inch O.D., thick-walled sampler. The sampler is lined with eighteen 2-3/8-inch I.D. brass rings. Relatively undisturbed, intact soil samples are retained in the brass rings.

S ("SPT") - a.k.a. Standard Penetration Test, an 18-inch-long, 2-inch O.D., 1-3/8-inch I.D. drive sampler.

B ("Bulk") - a.k.a. Bulk Sack Sample, a disturbed, but representative sample obtained from a specific depth interval placed in a large plastic bag.

PB ("Plastic Bag")- A disturbed, but representative sample obtained from a specific depth interval placed in a small sealable plastic bag.

(CONTINUED)

TerraCosta Consulting Group, Inc.
4455 Murphy Canyon Road, Suite 100
San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
MARINA PARK	2573	LEGEND

SITE LOCATION	START	FINISH	SHEET NO.
Newport Beach, CA	5/16/2008	5/16/2008	2 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Gregg Drilling	Hollow Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl B5	8	40		▼ n/a_

SAMPLING METHOD	NOTES
140-lb hammer / 30-inch drop	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
25									
30									
35									

KEY TO EXCAVATION LOGS

(CONTINUED)

NOTES ON FIELD INVESTIGATION

Borings were advanced using a truck-mounted Marl B5 drill rig with an 8-inch hollow-stem auger.

Standard Penetration Tests (SPT) and California Samplers were used to obtain soil samples. The SPT and California Samplers were driven into the soil at the bottom of the borings with a 140-pound hammer falling 30 inches. When the samplers were withdrawn from the boring, the samples were removed, visually classified, sealed in plastic containers, and taken to the laboratory for detailed inspection.

Free groundwater was encountered in the borings as shown on the logs.

Classifications are based upon the Unified Soil Classification System and include color, moisture, and consistency. Field descriptions have been modified to reflect results of laboratory inspection where deemed appropriate.

TerraCosta Consulting Group, Inc.
4455 Murphy Canyon Road, Suite 100
San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-1 b

LOG OF TEST BORING

PROJECT NAME: MARINA PARK		PROJECT NUMBER: 2573		BORING: **B-1**	
SITE LOCATION: Newport Beach, CA		START: 5/16/2008	FINISH: 5/16/2008	SHEET NO.: 1 of 2	
DRILLING COMPANY: Gregg Drilling	DRILLING METHOD: Hollow Stem Auger		LOGGED BY: G. Spaulding	CHECKED BY:	
DRILLING EQUIPMENT: Marl M5	BORING DIA. (in): 8	TOTAL DEPTH (ft): 31.5	GROUND ELEV (ft):	DEPTH/*ELEV.* GROUND WATER (ft): ▼ n/a_	
SAMPLING METHOD: 140-lb hammer / 30-inch drop	NOTES:				

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0									HYDRAULIC FILL **SAND to Silty SAND (SP/SM)** loose to medium dense, gray-brown, dry, with occasional shell fragments
									- Becomes medium dense, moist
5			1	25			SA		
10								▼	
			2	24			SA HD		BAY DEPOSITS **Medium SAND (SP/SM)** medium dense, gray, wet, with shell fragments
15			3	18			SA HD		

TerraCosta Consulting Group, Inc.
4455 Murphy Canyon Road, Suite 100
San Diego, California 92123

FIGURE A-2 a

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
MARINA PARK	2573	B-1

SITE LOCATION	START	FINISH	SHEET NO.
Newport Beach, CA	5/16/2008	5/16/2008	2 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Gregg Drilling	Hollow Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/ELEV. GROUND WATER (ft)
Marl M5	8	31.5		n/a

SAMPLING METHOD	NOTES
140-lb hammer / 30-inch drop	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			4	22			SA		
25			5	15			SA		
									OLDER ALLUVIAL DEPOSITS **Coarse SAND (SP-SM)** medium dense, gray, wet
30			6	37					
									Boring terminated at depth of 31.5 feet. *Groundwater encountered at approximately 10 feet at time of excavation.*
35									

TCG_METRIC_LOG(3) 2573.GPJ GDCLOGMT.GDT 8/7/08

TerraCosta Consulting Group, Inc.
4455 Murphy Canyon Road, Suite 100
San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-2 b

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
MARINA PARK	2573	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
Newport Beach, CA	5/16/2008	5/16/2008	1 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Gregg Drilling	Hollow Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl B5	8	31.5		⊻ n/a_

SAMPLING METHOD	NOTES
140-lb hammer / 30-inch drop	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
									HYDRAULIC FILL **Medium SAND (SP-SM)** loose to medium dense, gray-brown, damp to wet
									- Becomes medium dense, moist
5			1	7			SA		
								⊻	- Becomes wet
									- Becomes coarse with approximately 10 percent shell fragments
10			2	20			SA		
									BAY DEPOSITS **Medium SAND (SP-SM)** medium dense, gray, et
15			3	13					

TerraCosta Consulting Group, Inc.
4455 Murphy Canyon Road, Suite 100
San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-3 a

LOG OF TEST BORING

PROJECT NAME	PROJECT NUMBER	BORING
MARINA PARK	2573	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
Newport Beach, CA	5/16/2008	5/16/2008	2 of 2

DRILLING COMPANY	DRILLING METHOD	LOGGED BY	CHECKED BY
Gregg Drilling	Hollow Stem Auger	G. Spaulding	

DRILLING EQUIPMENT	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GROUND WATER (ft)
Marl B5	8	31.5		⊻ n/a_

SAMPLING METHOD	NOTES
140-lb hammer / 30-inch drop	

DEPTH (ft)	ELEVATION (ft)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS/ft)	DRY DENSITY (pcf)	MOISTURE (%)	OTHER TESTS	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			4	14			SA		
25									
			5	50/3"					- Sampler on rock
									<u>OLDER ALLUVIAL DEPOSITS</u> **Coarse SAND (SP-SM)** medium dense, red-brown, wet
30									
			6	38			SA		
									Boring terminated at depth of 31.5 feet. *Groundwater encountered at depth of approximately 6.5 feet at time of excavation.*
35									

TCG_METRIC_LOG(3) 2573.GPJ GDCLOGMGT.GDT 8/7/08

TerraCosta Consulting Group
TerraCosta Consulting Group, Inc.
4455 Murphy Canyon Road, Suite 100
San Diego, California 92123

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED.

FIGURE A-3 b

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

May 19, 2008

Terra Costa Consulting Group
Attn: Bob Smille
4455 Murphy Canyon Road
San Diego, CA 92123

Subject: CPT Site Investigation
Marina Park
Balboa Peninsula, California
GREGG Project Number: 08-206SH

Dear Mr. Smille:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	Resistivity Cone Penetration Tests	(RCPTU)	☐
5	UVOST Laser Induced Fluorescence	(UVOST)	☐
6	Groundwater Sampling	(GWS)	☐
7	Soil Sampling	(SS)	☐
8	Vapor Sampling	(VS)	☐
9	Vane Shear Testing	(VST)	☐
10	SPT Energy Calibration	(SPTE)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (562) 427-6899.

Sincerely,
GREGG Drilling & Testing, Inc.

Peter Robertson
Technical Operations

2726 Walnut Ave • Signal Hill, California 90755 • (562) 427-6899 • FAX (562) 427-3314
OTHER OFFICES: SAN FRANCISCO • HOUSTON • SOUTH CAROLINA
www.greggdrilling.com

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (Feet)	Depth of Groundwater Samples (Feet)	Depth of Soil Samples (Feet)	Depth of Pore Pressure Dissipation Tests (Feet)
CPT-01	5/16/08	50	-	-	-
CPT-02	5/16/08	50	-	-	-
CPT-03	5/16/08	50	-	-	-
CPT-04	5/16/08	30	-	-	-
CPT-05	5/16/08	34	-	-	-
CPT-06	5/16/08	50	-	-	-
CPT-07	5/16/08	35	-	-	-
CPT-08	5/16/08	30	-	-	-
CPT-09	5/16/08	30	-	-	-
CPT-10	5/16/08	30	-	-	22
CPT-11	5/16/08	47	-	-	-
CPT-12	5/16/08	43	-	-	-

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*. The soundings were conducted using a 20 ton capacity cone with a tip area of 15 cm^2 and a friction sleeve area of 225 cm^2. The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and penetration pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), *Figure CPT*. It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain penetration pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

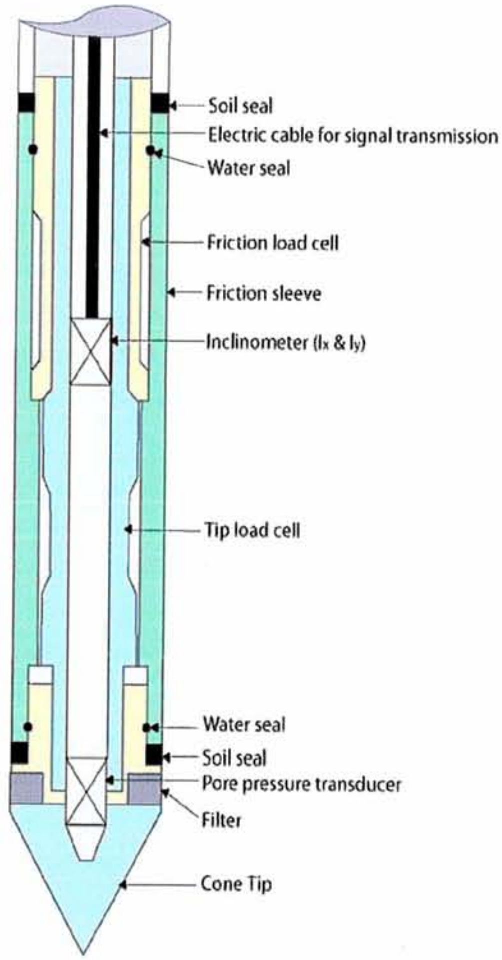


Figure CPT

When the soundings are complete, the test holes are grouted using a Gregg support rig. The grouting procedures generally consist of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

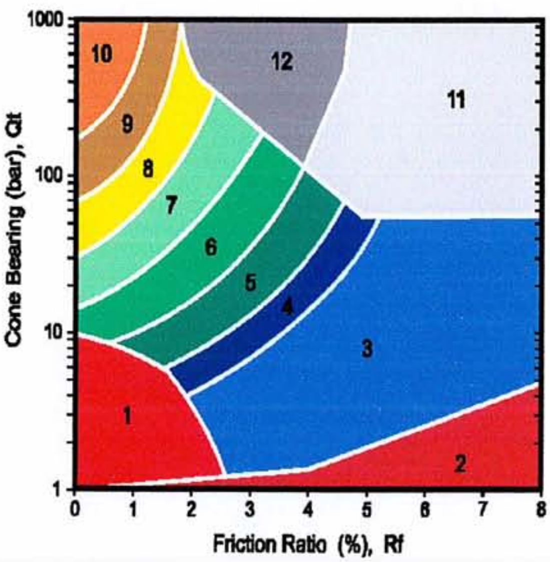
Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



(After Robertson, et al., 1986)

ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

*over consolidated or cemented

Figure SBT

Pore Pressure Dissipation Tests (PPDT)

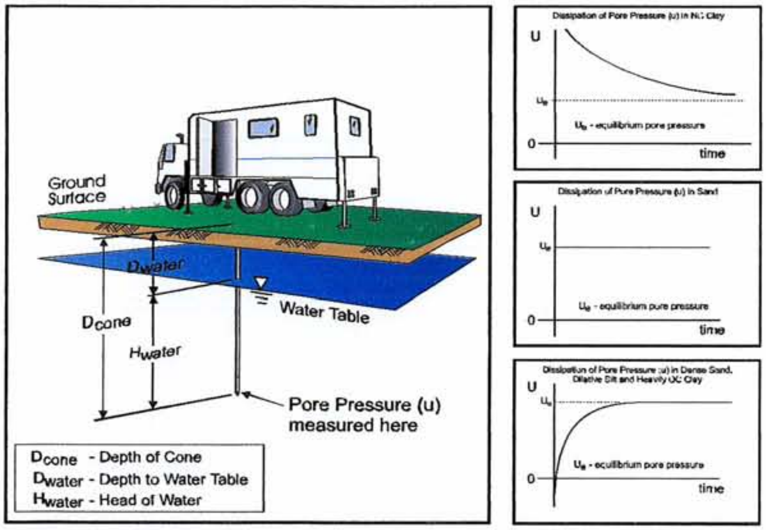
Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals measured hydrostatic water pressures and determined the approximate depth of the ground water table. A PPDT is conducted when the cone is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded by a computer system.
Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time, *Figure PPDT*. This time is commonly referred to as t_{100}, the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1992.

A summary of the pore pressure dissipation tests is summarized in Table 1.



Water Table Calculation

$$D_{water} = D_{cone} - H_{water}$$

where $H_{water} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
1tsf = 0.958 bar = 13.9 psi
1m = 3.28 feet

Figure PPDT

Bibliography

Lunne, T., Robertson, P.K. and Powell, J.J.M., "Cone Penetration Testing in Geotechnical Practice" E & FN Spon. ISBN 0 419 23750, 1997

Roberston, P.K., "Soil Classification using the Cone Penetration Test", Canadian Geotechnical Journal, Vol. 27, 1990 pp. 151-158.

Mayne, P.W., "NHI (2002) Manual on Subsurface Investigations: Geotechnical Site Characterization", available through www.ce.gatech.edu/~geosys/Faculty/Mayne/papers/index.html, Section 5.3, pp. 107-112.

Robertson, P.K., R.G. Campanella, D. Gillespie and A. Rice, "Seismic CPT to Measure In-Situ Shear Wave Velocity", Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8, 1986 pp. 791-803.

Robertson, P.K., Sully, J., Woeller, D.J., Lunne, T., Powell, J.J.M., and Gillespie, D.J., "Guidelines for Estimating Consolidation Parameters in Soils from Piezocone Tests", Canadian Geotechnical Journal, Vol. 29, No. 4, August 1992, pp. 539-550.

Robertson, P.K., T. Lunne and J.J.M. Powell, "Geo-Environmental Application of Penetration Testing", Geotechnical Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Campanella, R.G. and I. Weemees, "Development and Use of An Electrical Resistivity Cone for Groundwater Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegger, "Reliability of Soil Gas Sampling and Characterization Techniques", International Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants Using the UVIF-CPT", 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

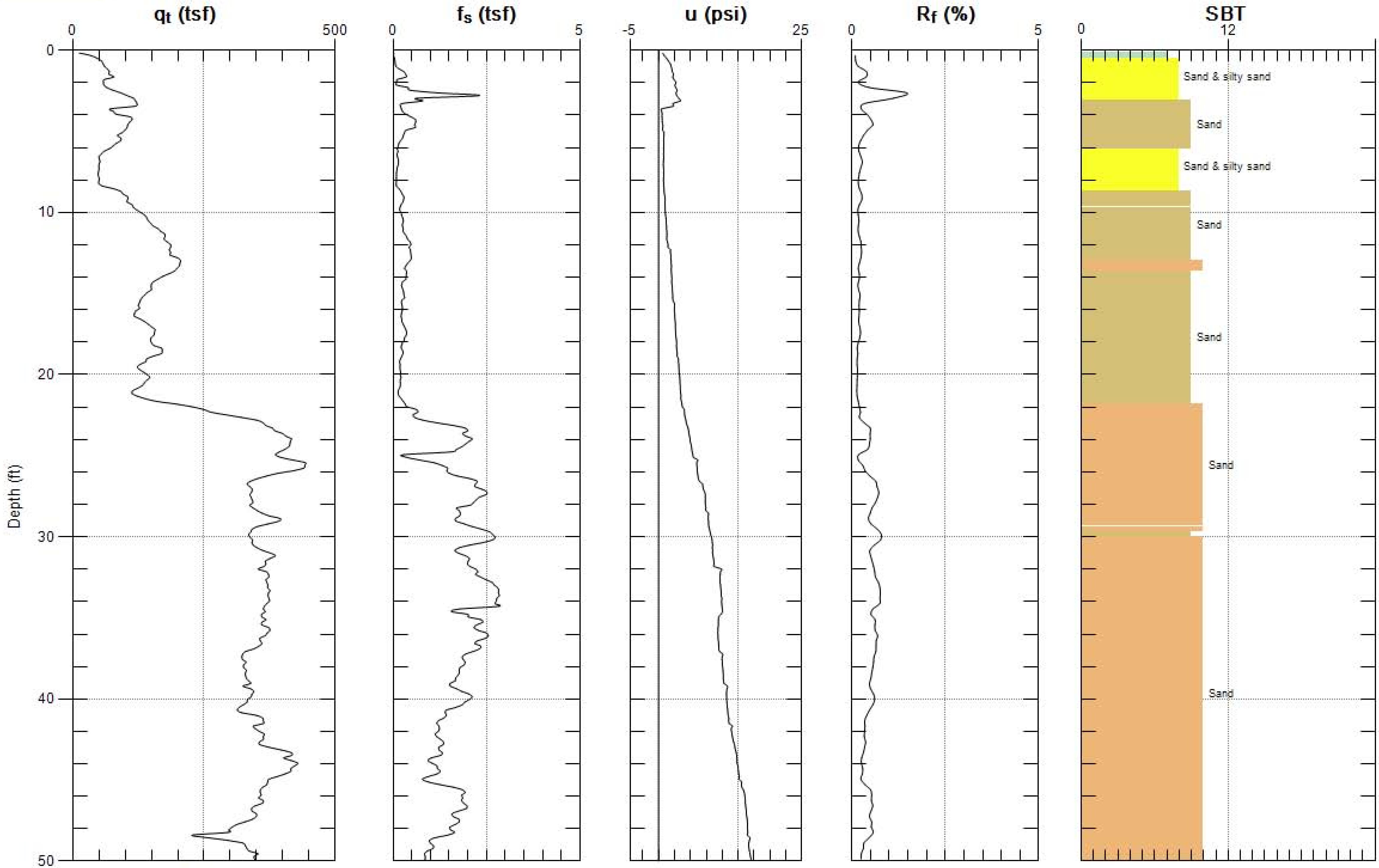
Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from Discrete-Depth Groundwater Samplers" BAT EnviroProbe and QED HydroPunch, Sixth national Outdoor Action Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-01

Engineer: B. SMILLE
Date: 5/16/2008 07:10



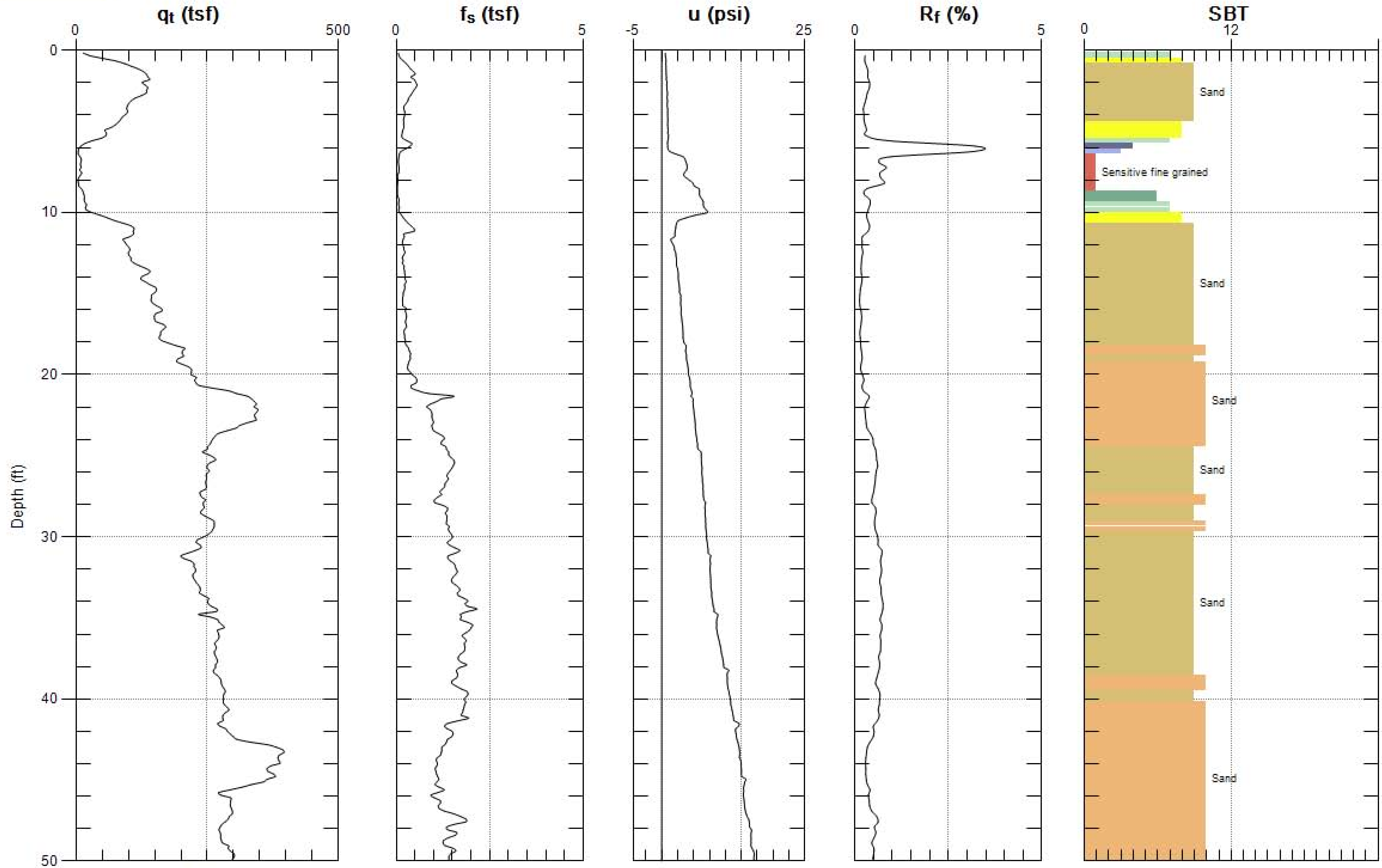
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA COSTA**

Site: MARINA PARK
Sounding: CPT-02

Engineer: B. SMILLE
Date: 5/16/2008 08:02



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

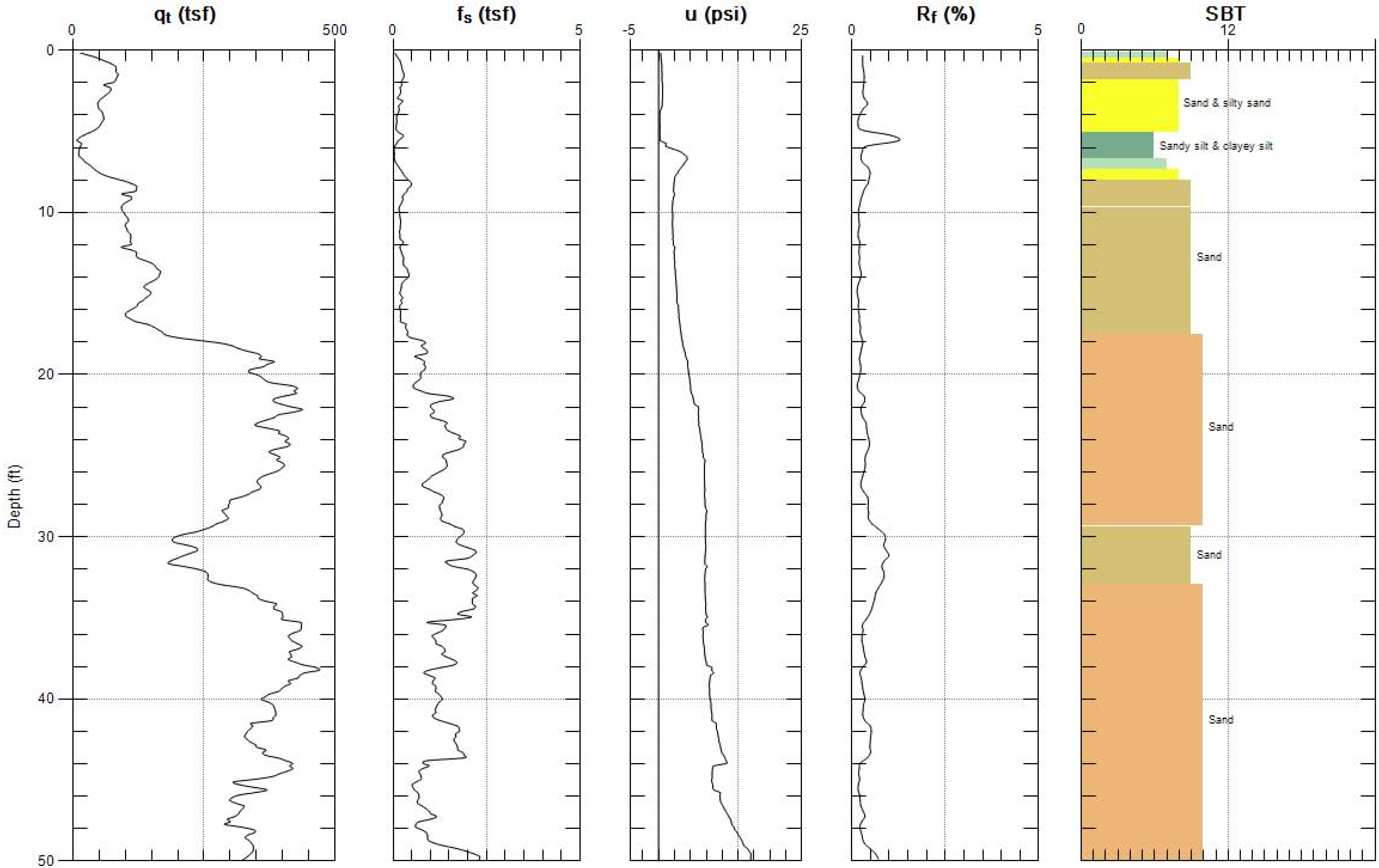
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-03

Engineer: B. SMILLE

Date: 5/16/2008 08:24



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

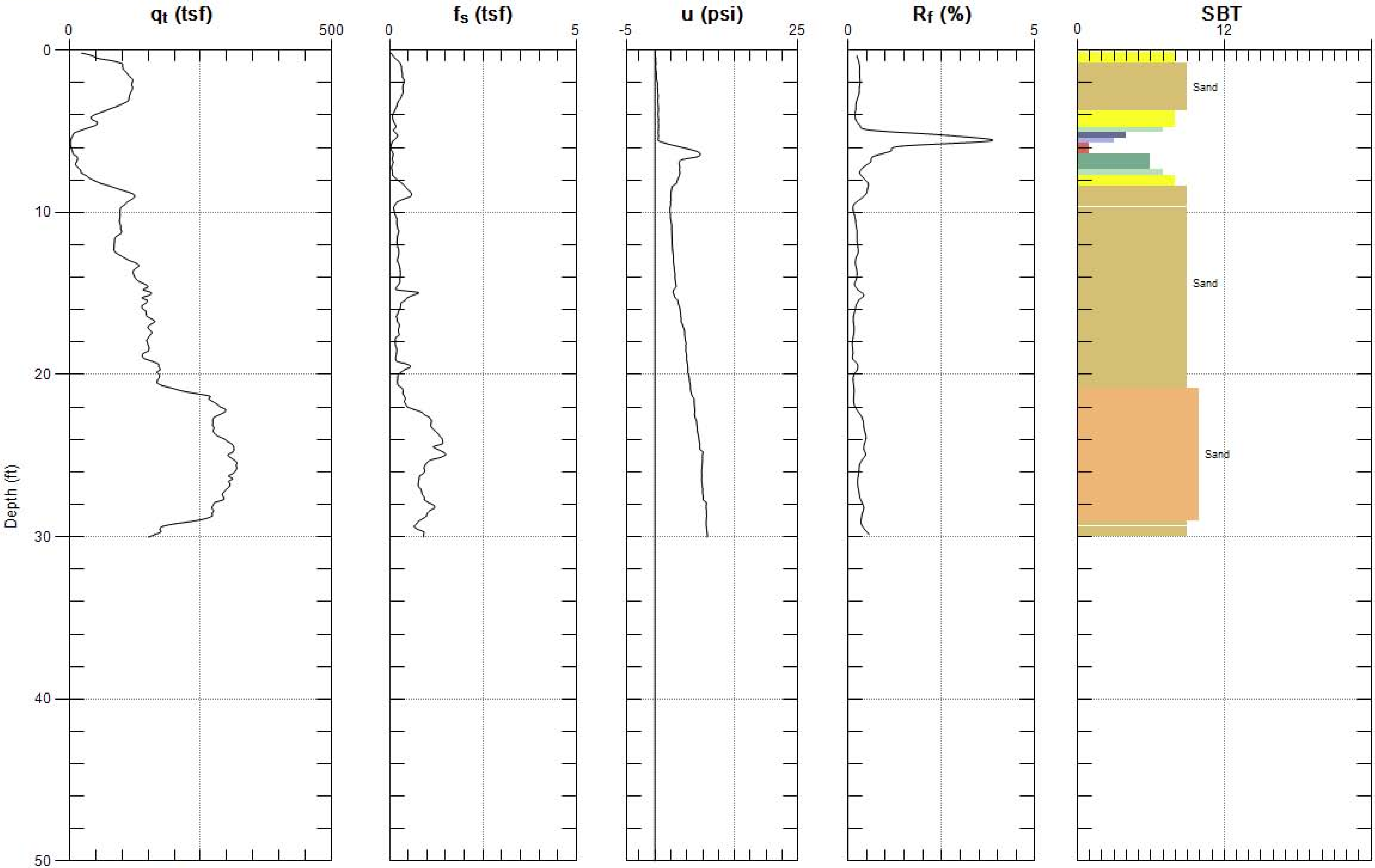
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-04

Engineer: B. SMILLE

Date: 5/16/2008 08:55



Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

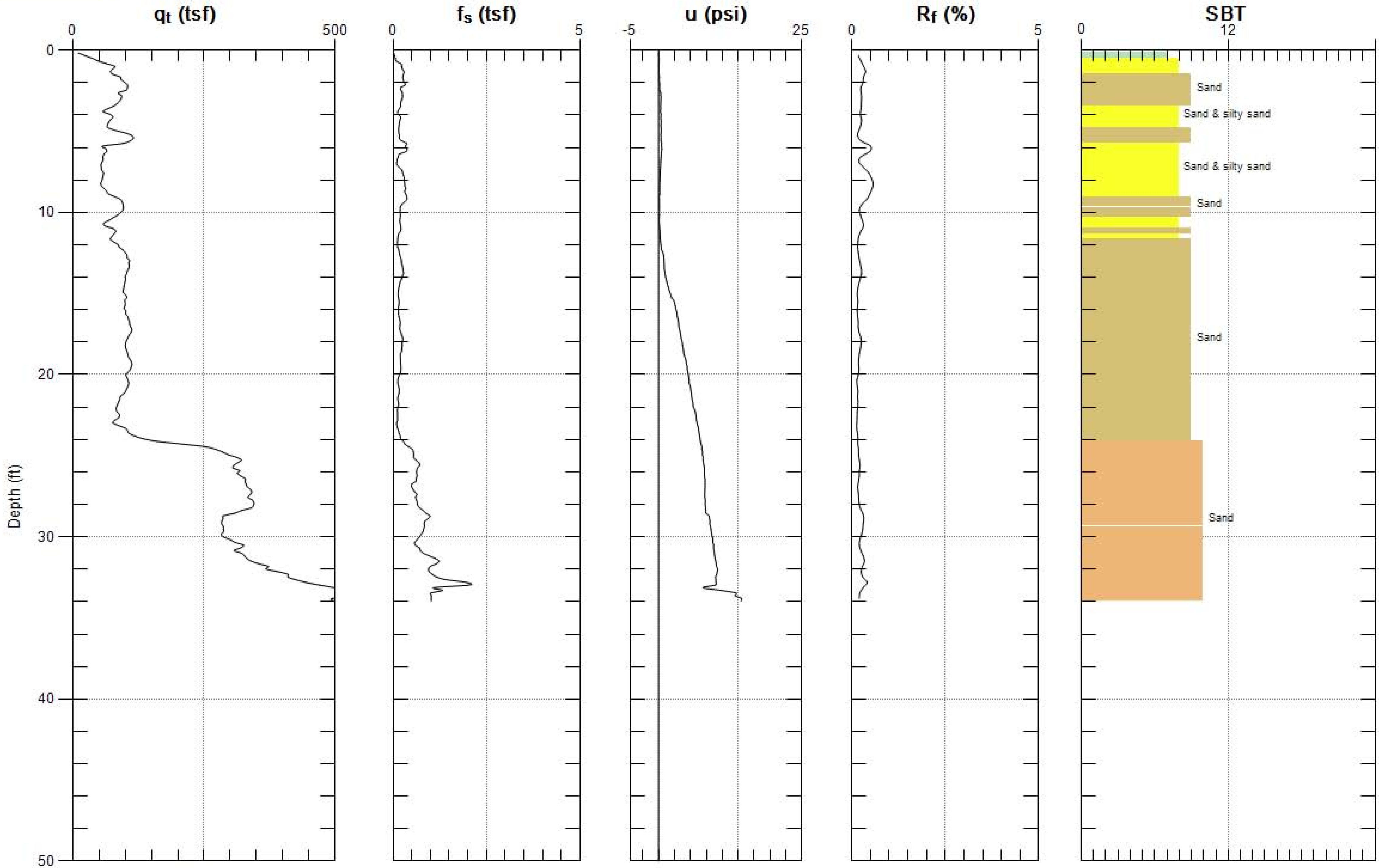
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-05

Engineer: B. SMILLE

Date: 5/16/2008 09:14



Max. Depth: 33.957 (ft)
Avg. Interval: 0.328 (ft)

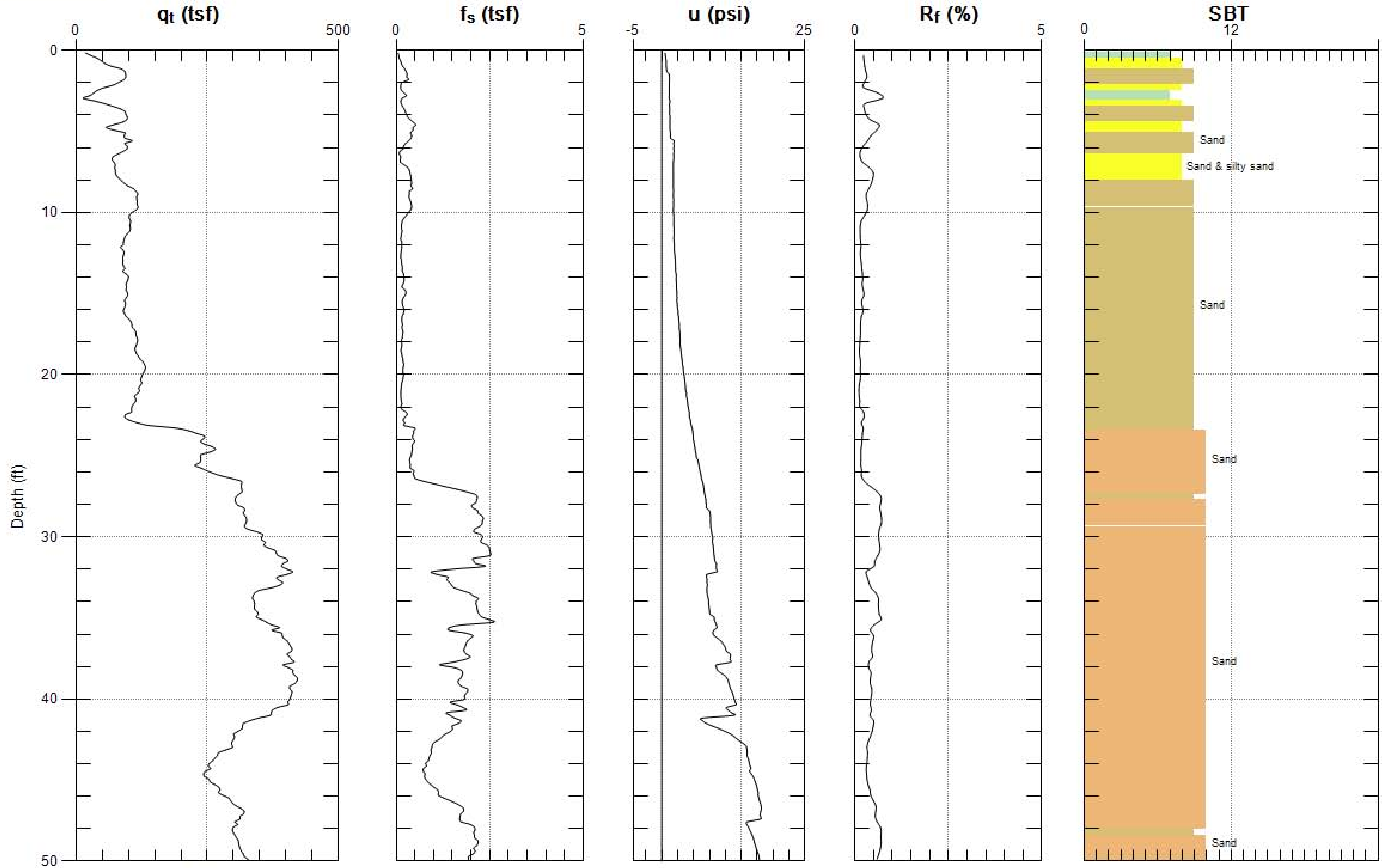
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-06

Engineer: B. SMILLE
Date: 5/16/2008 09:33



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

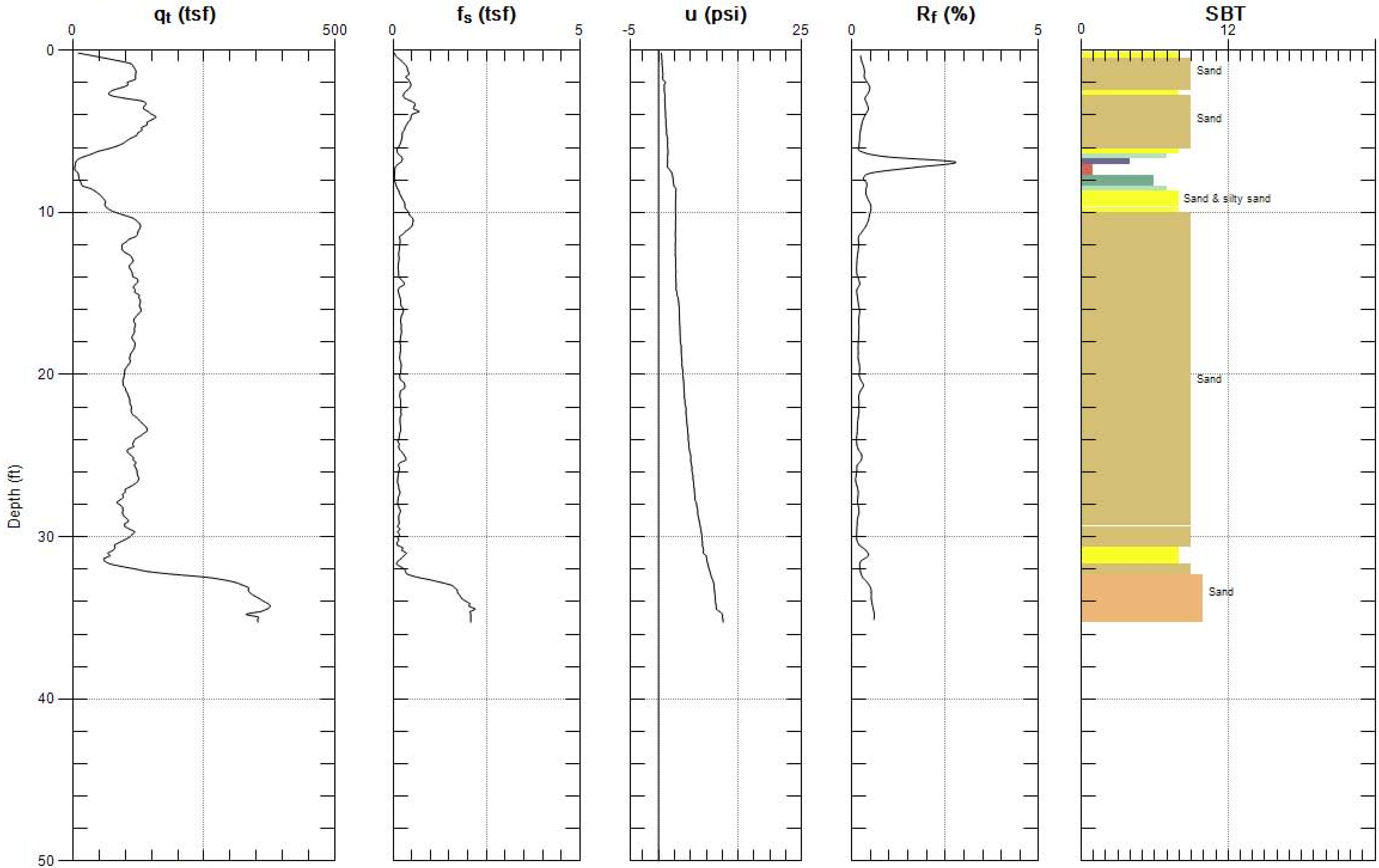
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-07

Engineer: B. SMILLE
Date: 5/16/2008 10:30



Max. Depth: 35.269 (ft)
Avg. Interval: 0.328 (ft)

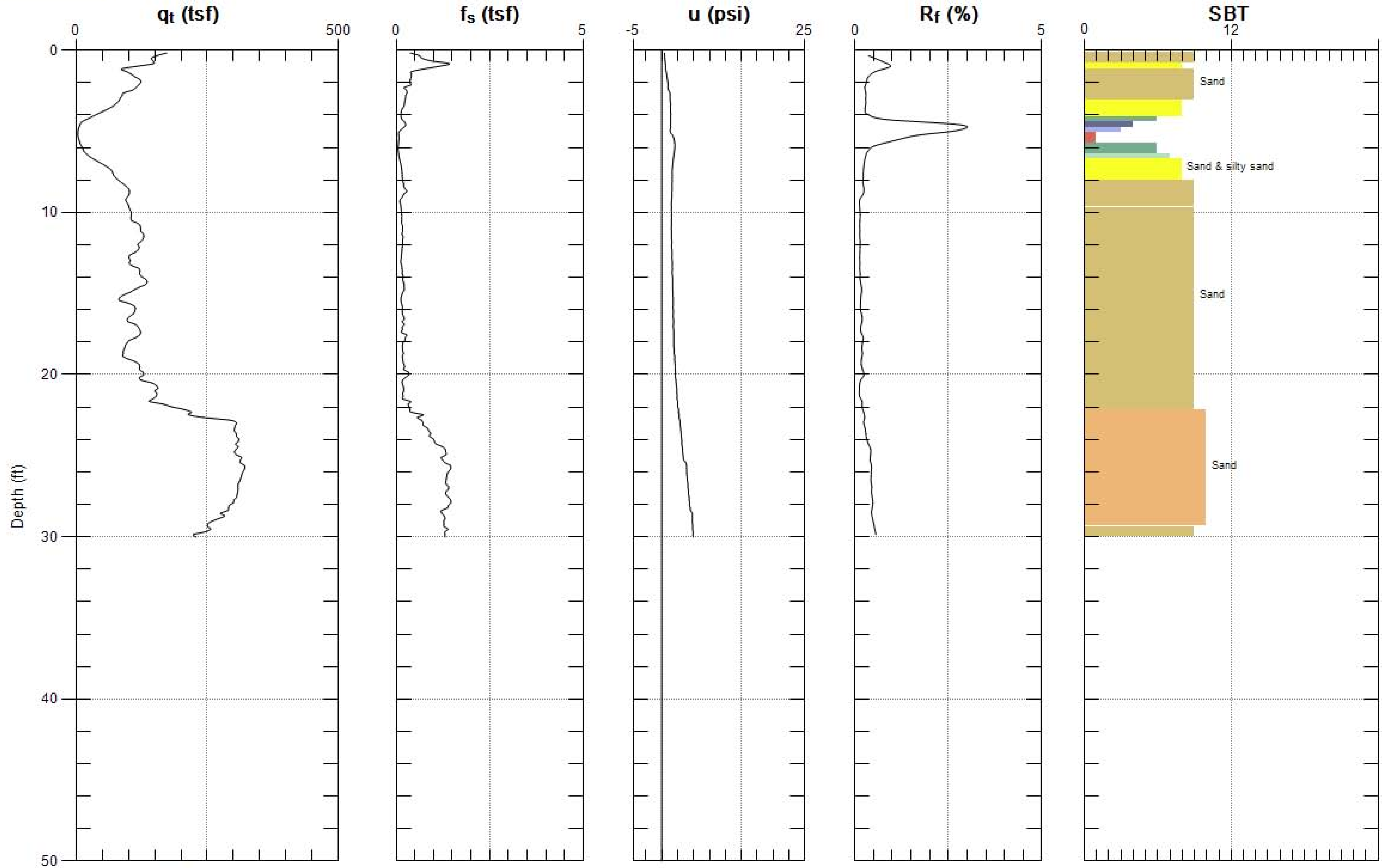
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-08

Engineer: B. SMILLE
Date: 5/16/2008 10:59



Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

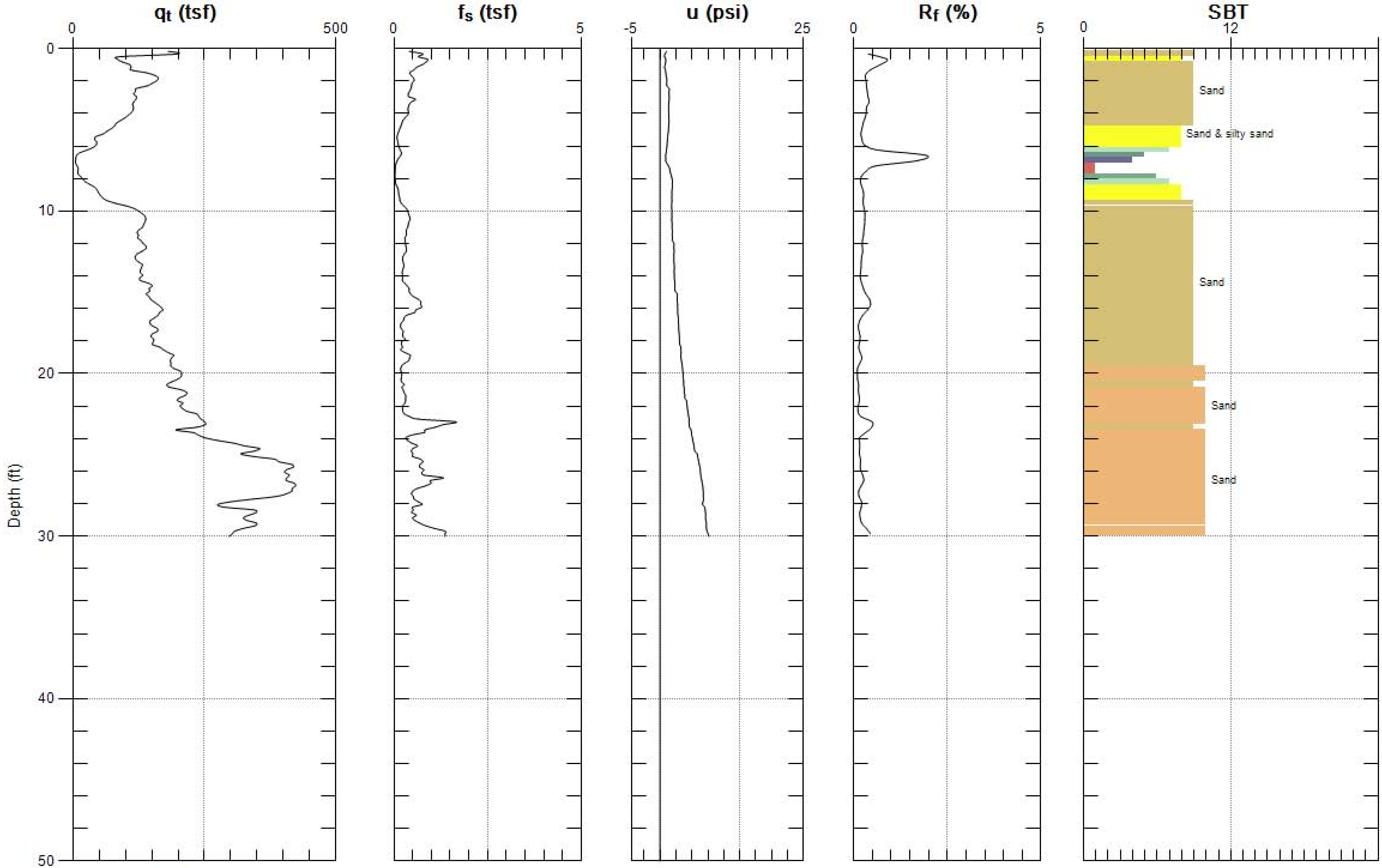
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-09

Engineer: B. SMILLE
Date: 5/16/2008 11:22



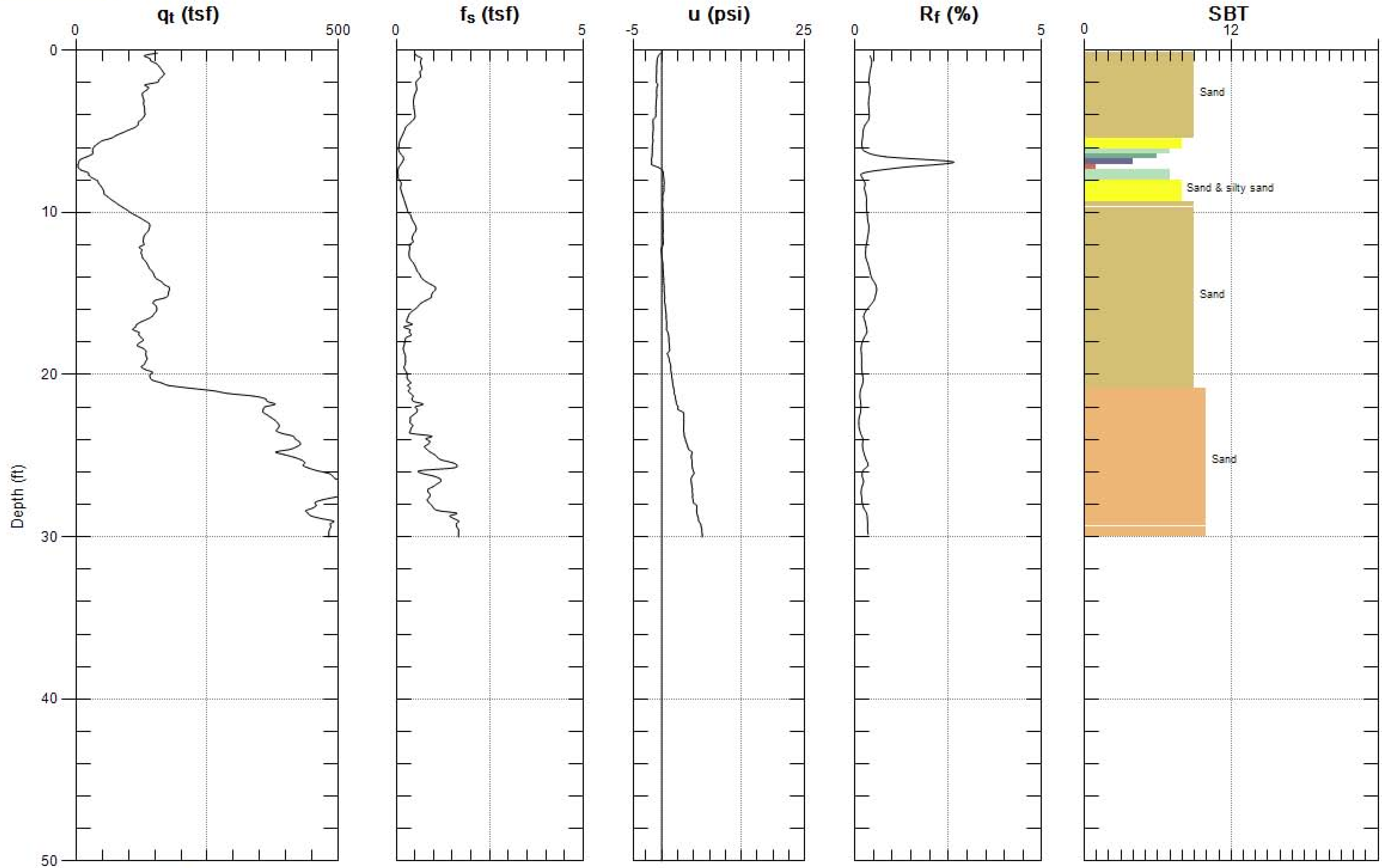
Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-10

Engineer: B. SMILLE
Date: 5/16/2008 11:39



Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

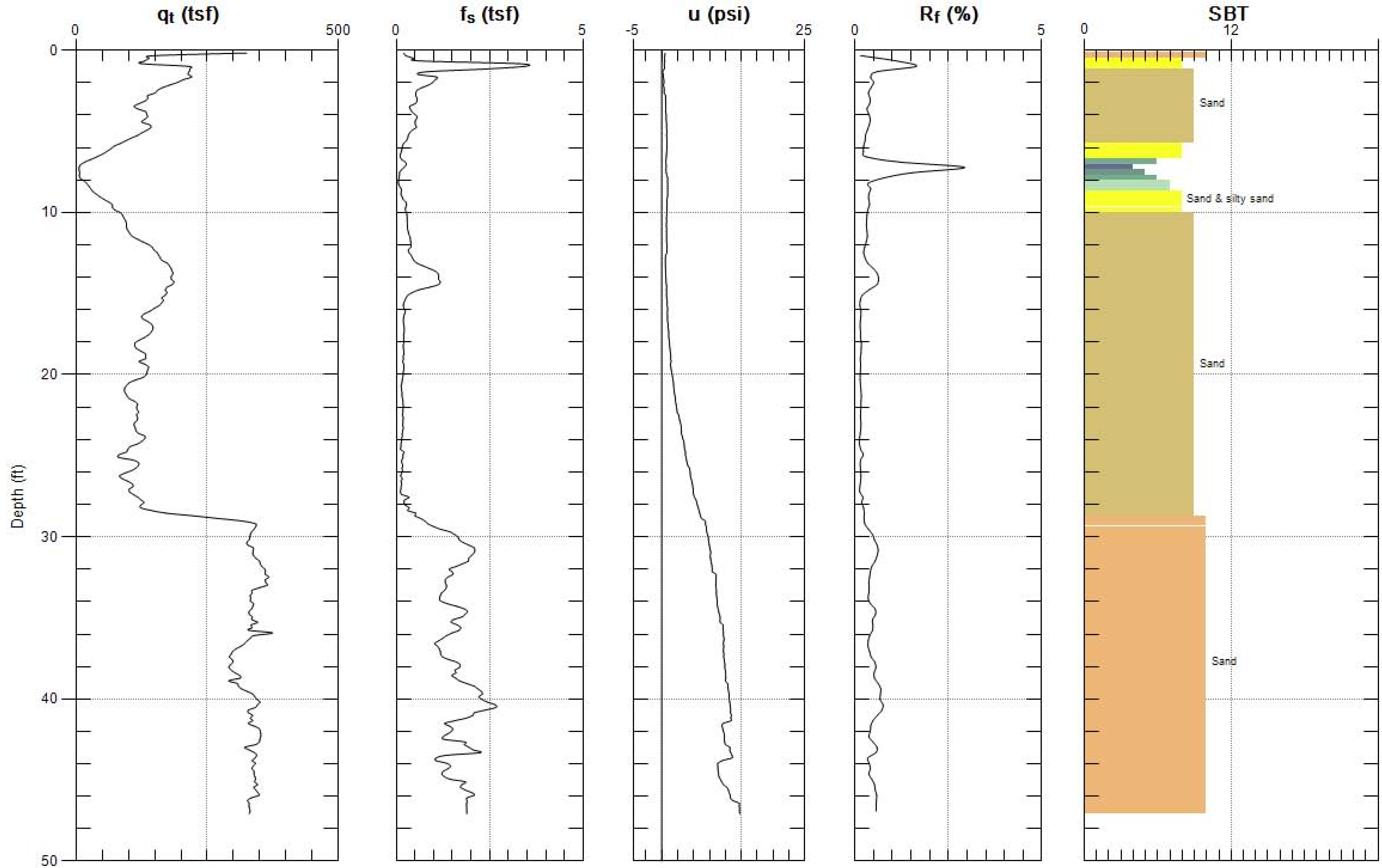
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-11

Engineer: B. SMILLE

Date: 5/16/2008 12:10



Max. Depth: 47.080 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

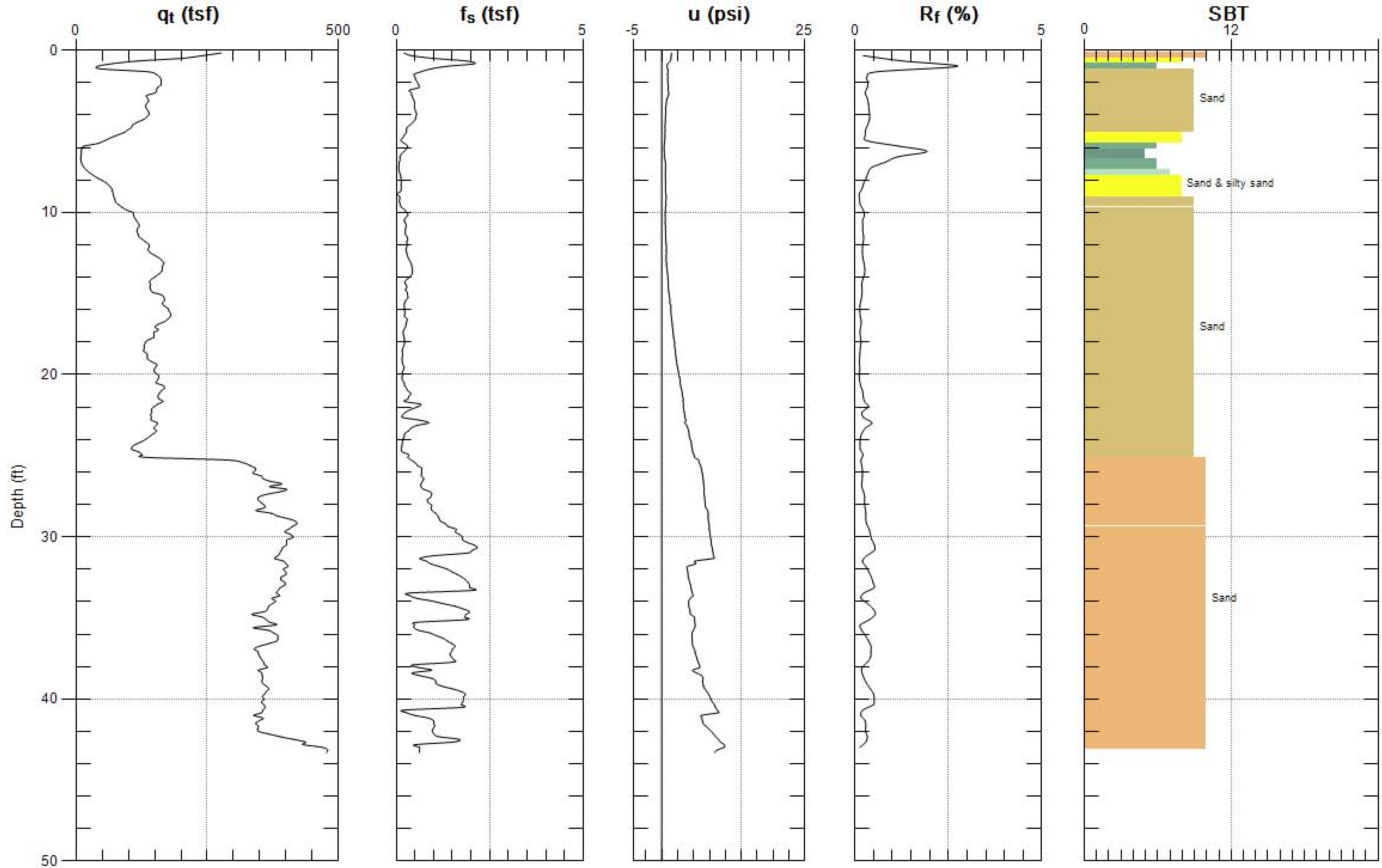
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-12

Engineer: B. SMILLE

Date: 5/16/2008 12:31



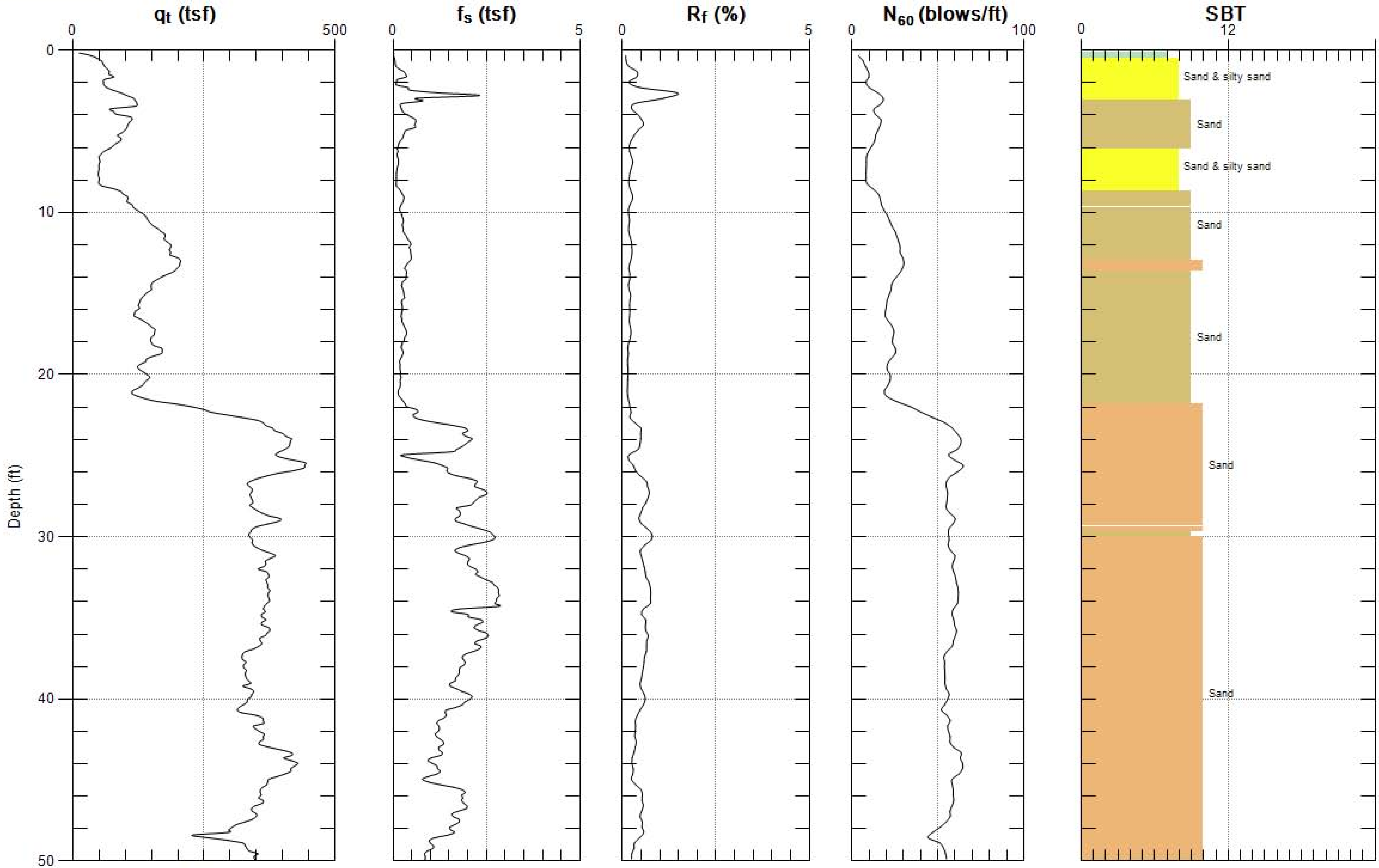
Max. Depth: 43.307 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA COSTA**

Site: MARINA PARK
Sounding: CPT-01

Engineer: B. SMILLE
Date: 5/16/2008 07:10



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

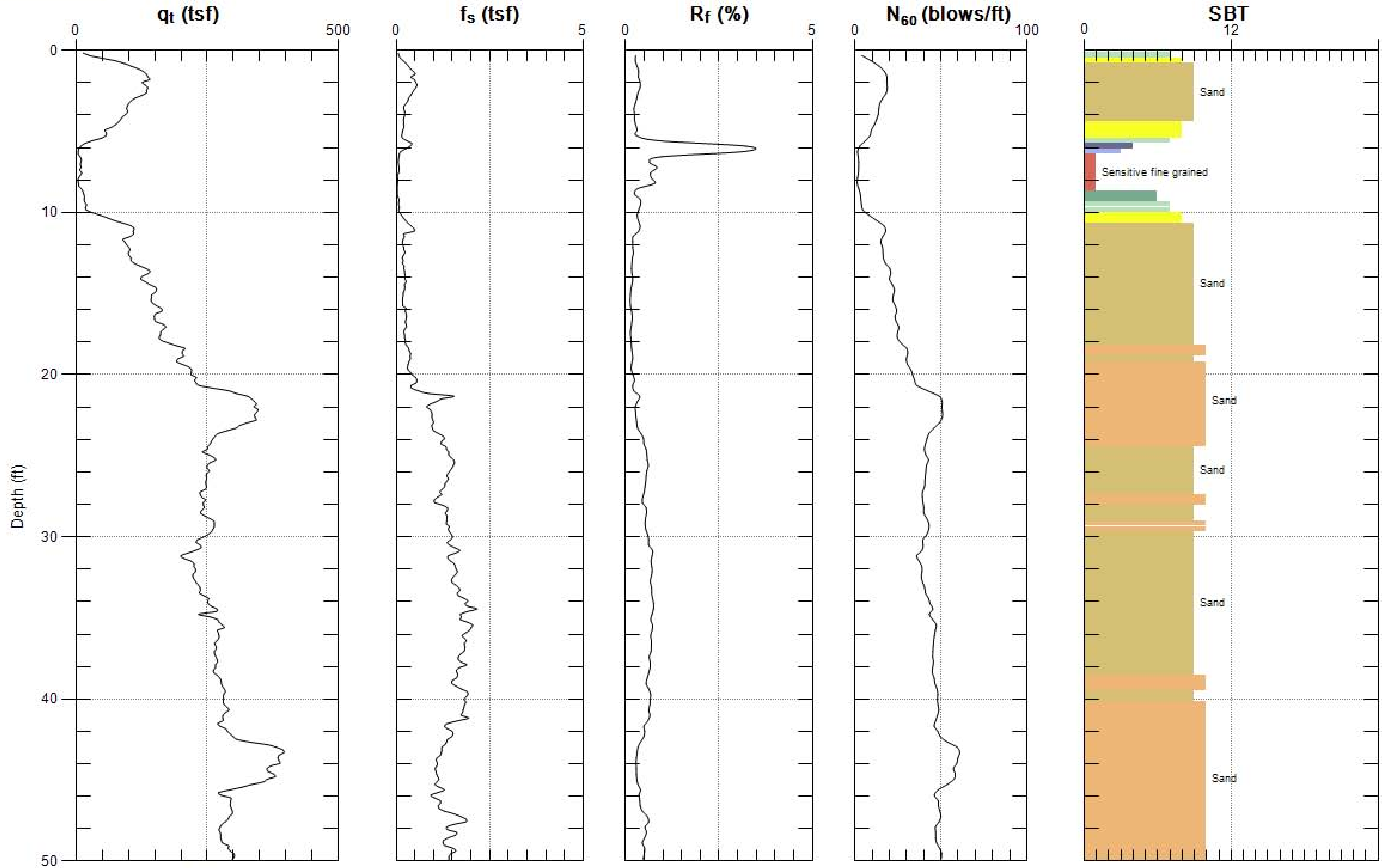
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-02

Engineer: B. SMILLE

Date: 5/16/2008 08:02



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

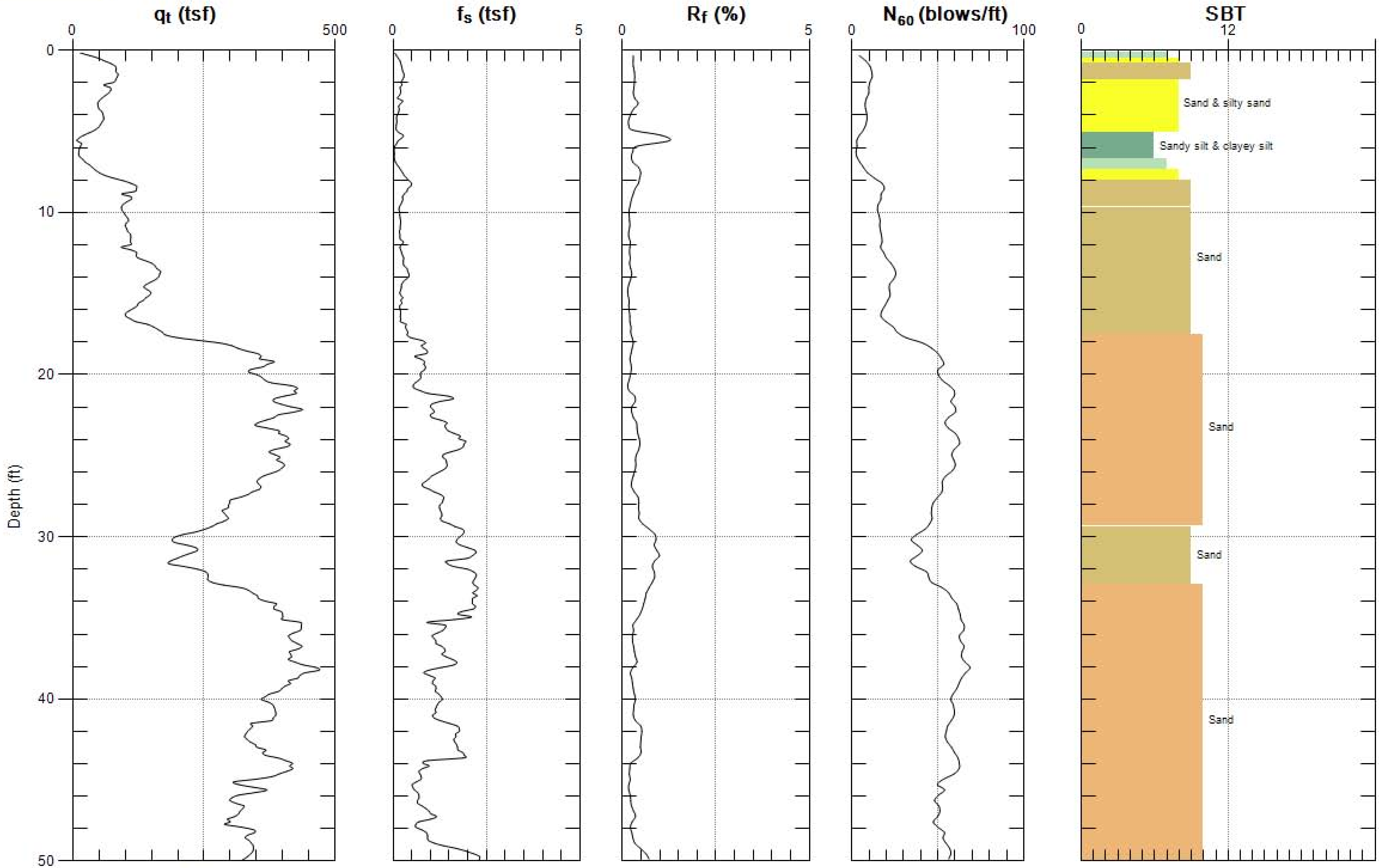
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-03

Engineer: B. SMILLE

Date: 5/16/2008 08:24



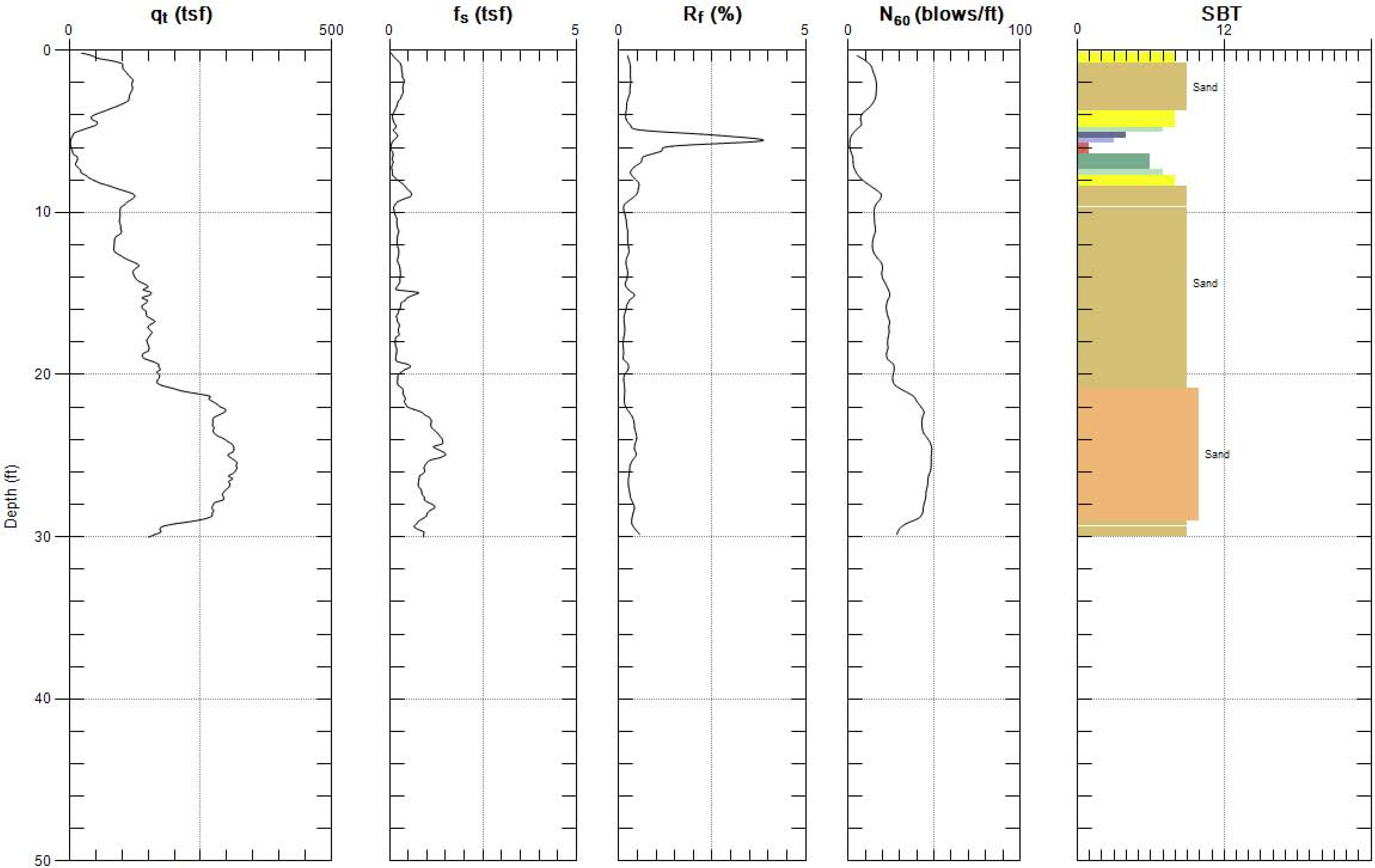
Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA COSTA

Site: MARINA PARK | Engineer: B. SMILLE
Sounding: CPT-04 | Date: 5/16/2008 08:55



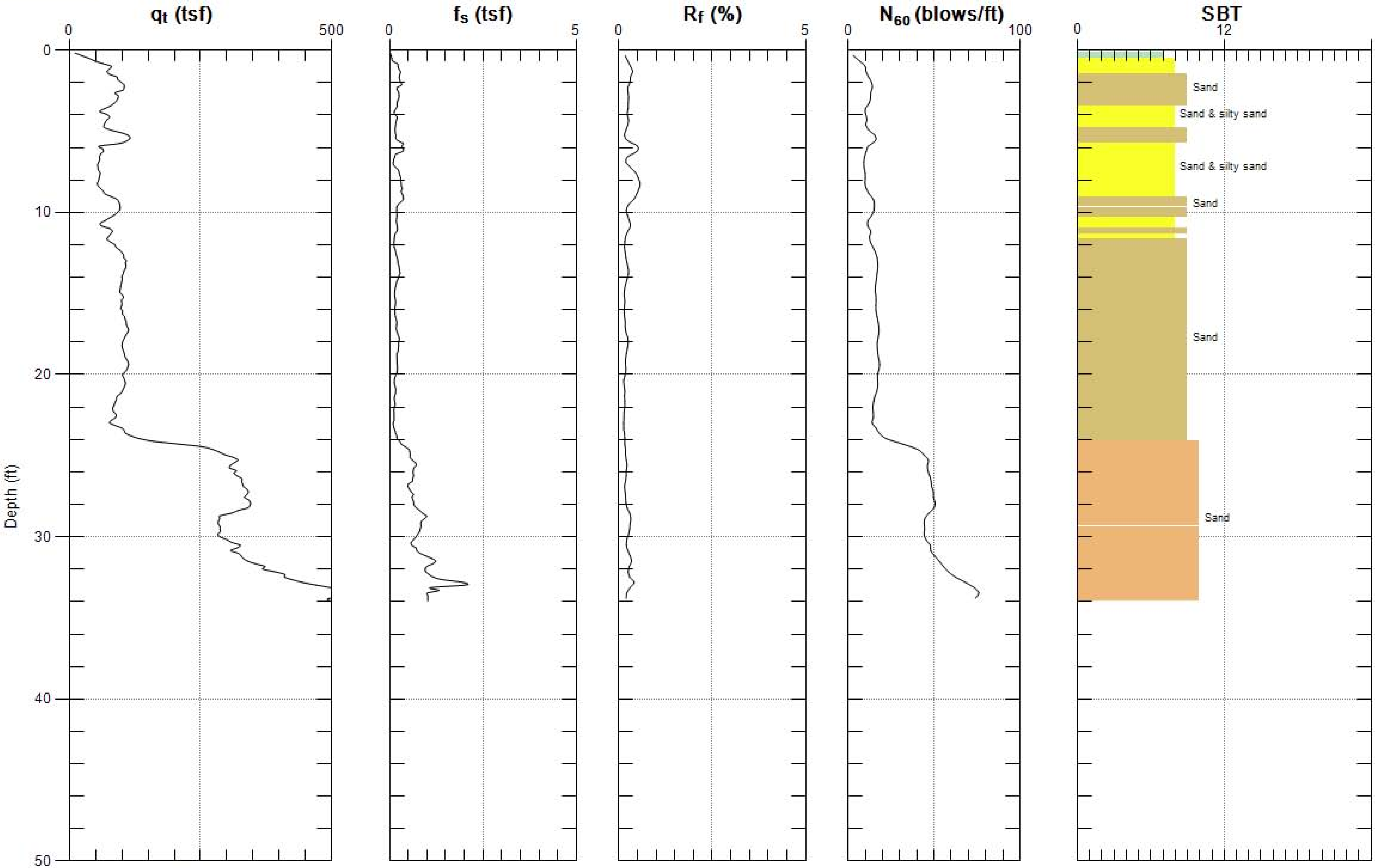
Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **TERRA COSTA**

Site: MARINA PARK
Sounding: CPT-05

Engineer: B. SMILLE
Date: 5/16/2008 09:14



Max. Depth: 33.957 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

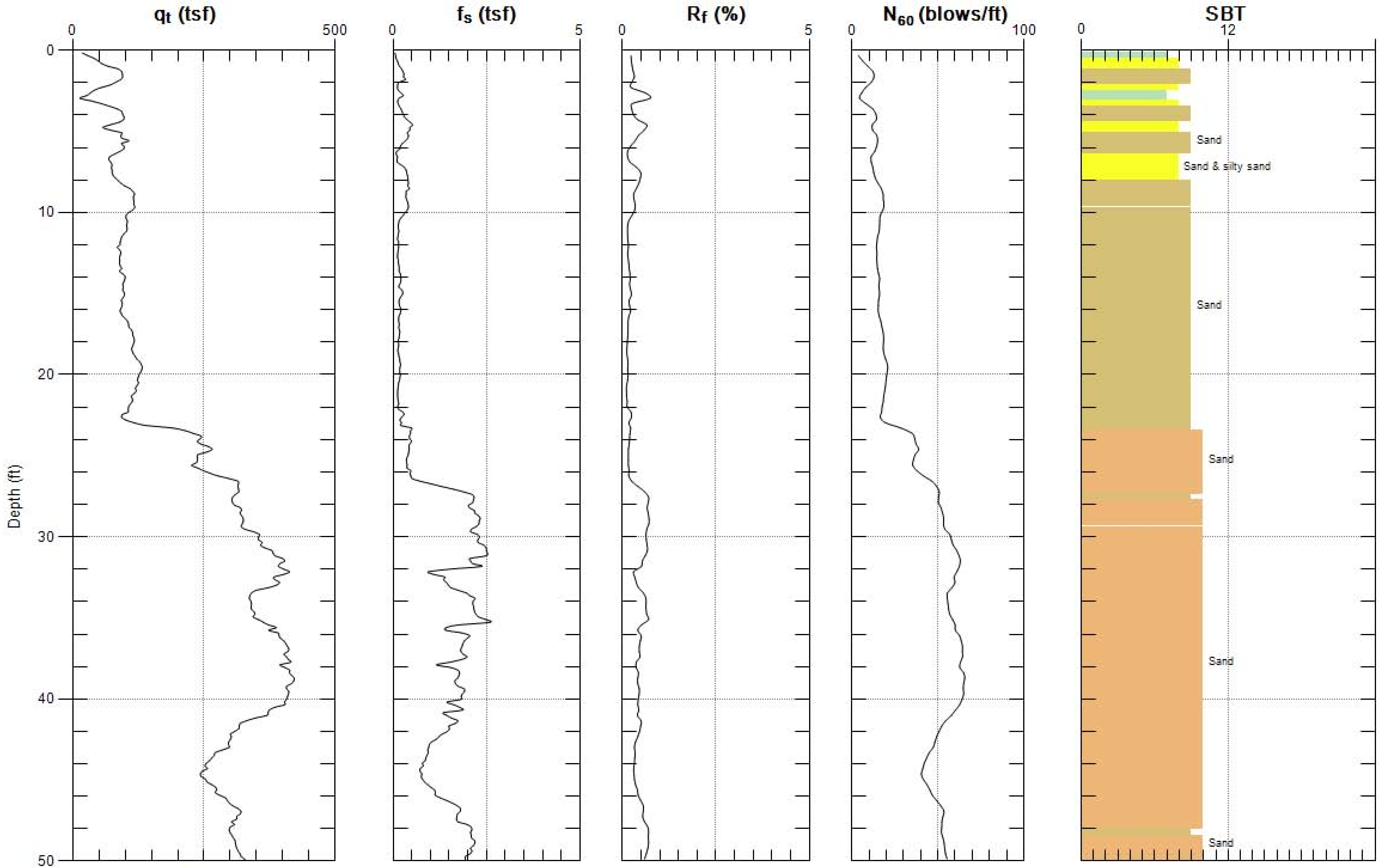
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-06

Engineer: B. SMILLE

Date: 5/16/2008 09:33



Max. Depth: 50.033 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

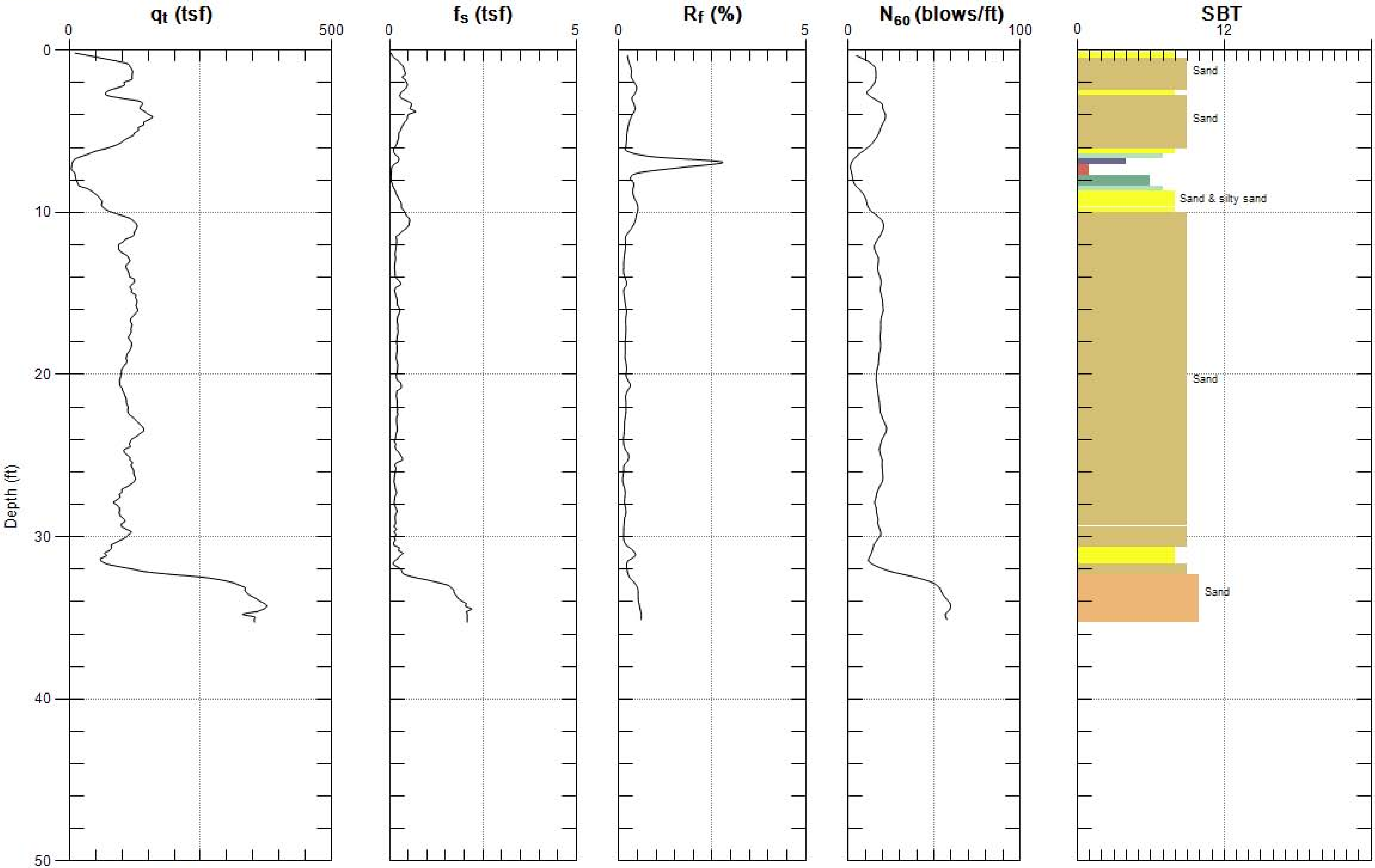
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-07

Engineer: B. SMILLE

Date: 5/16/2008 10:30



Max. Depth: 35.269 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

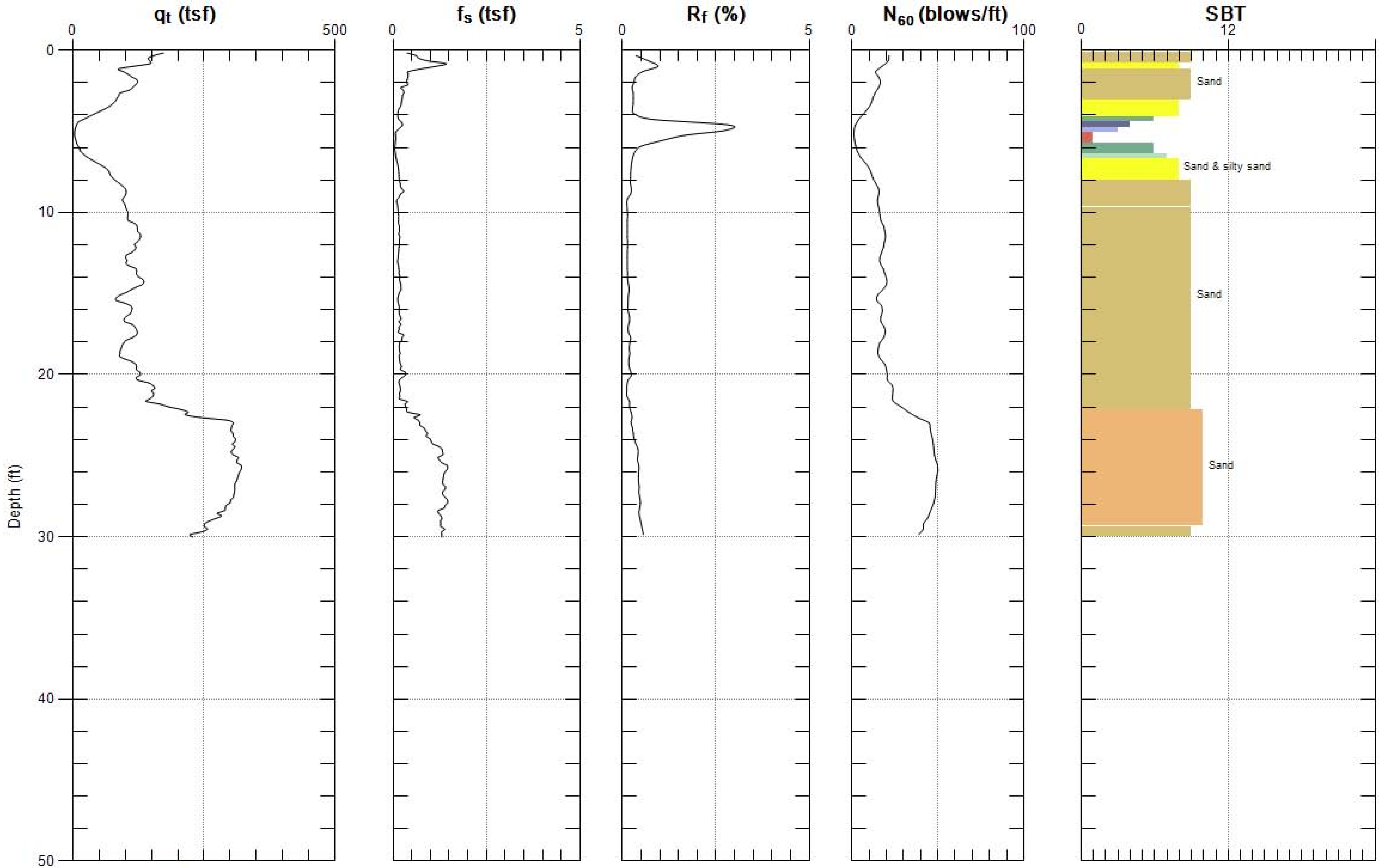
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-08

Engineer: B. SMILLE

Date: 5/16/2008 10:59



Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

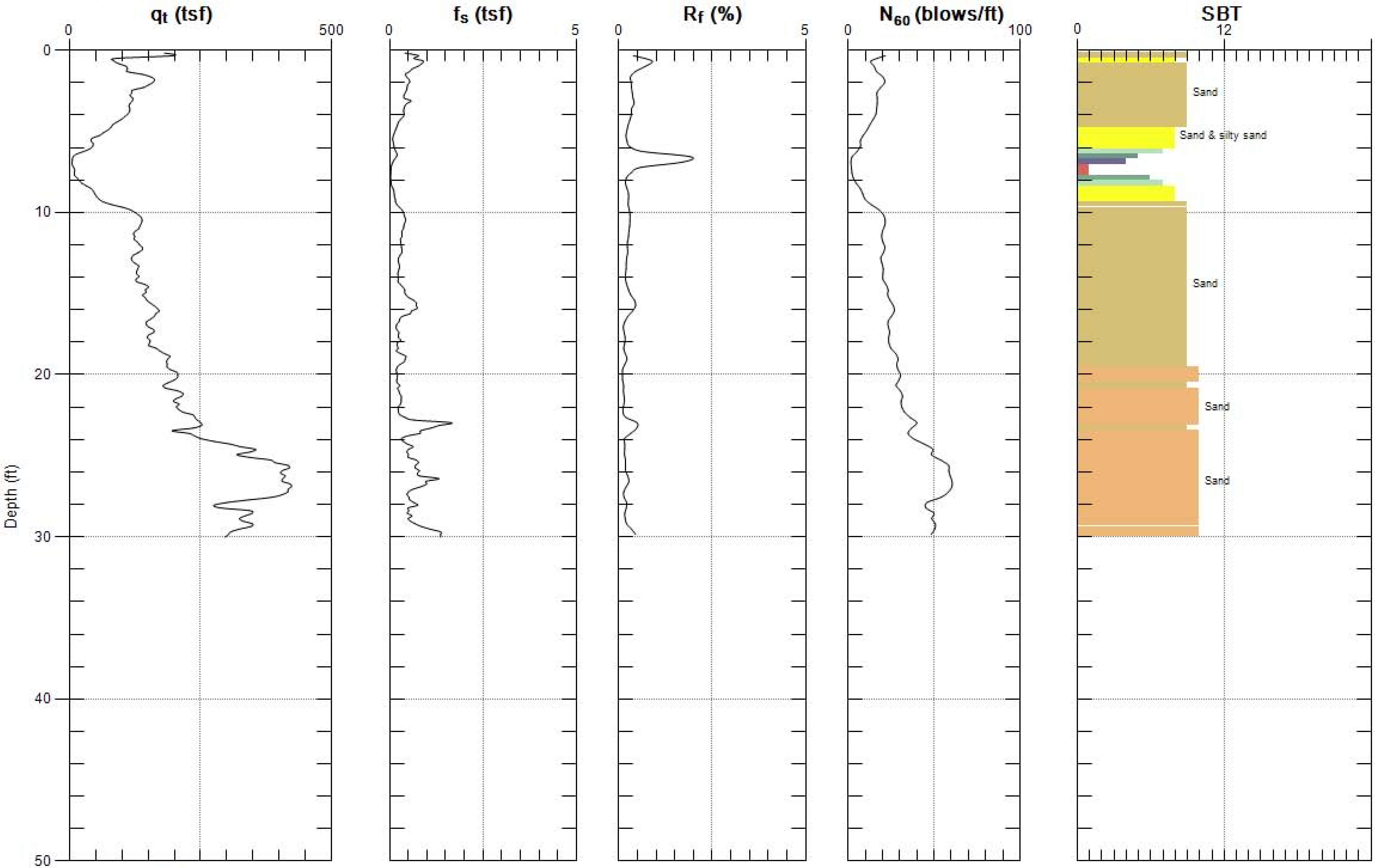
SBT: Soil Behavior Type (Robertson 1990)

GREGG

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-09

Engineer: B. SMILLE
Date: 5/16/2008 11:22



Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

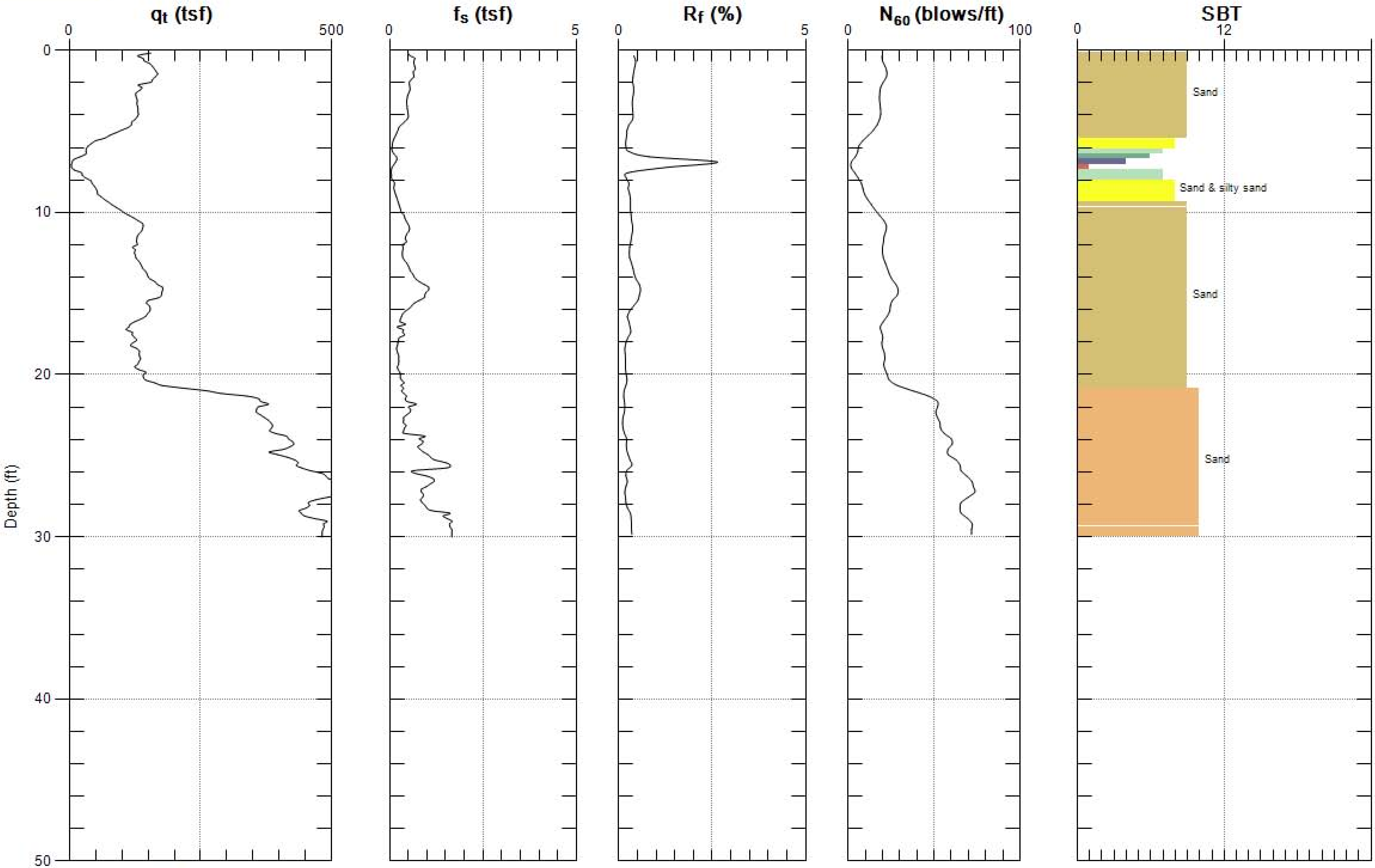
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-10

Engineer: B. SMILLE

Date: 5/16/2008 11:39



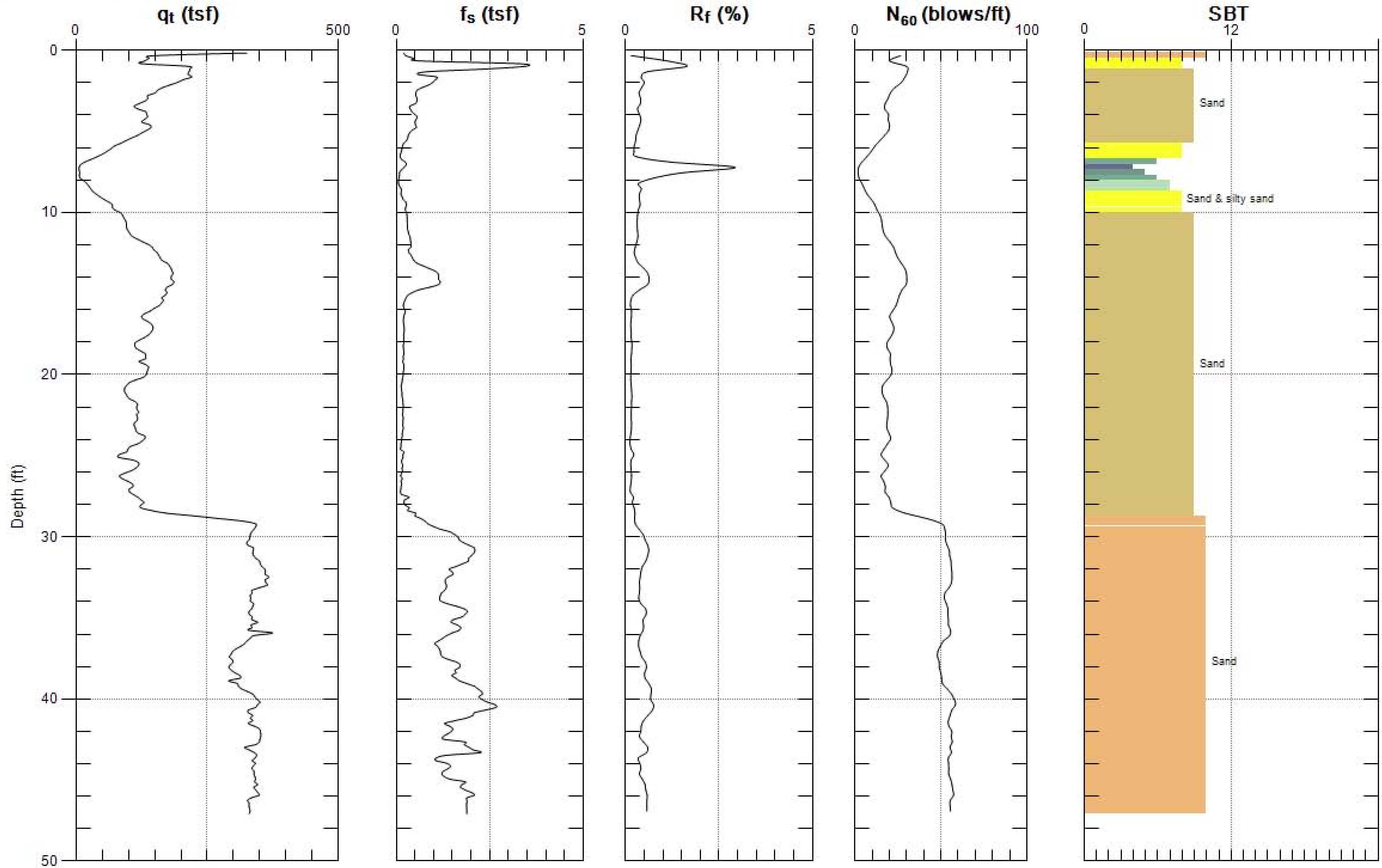
Max. Depth: 30.020 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

TERRA COSTA

Site: MARINA PARK
Sounding: CPT-11

Engineer: B. SMILLE
Date: 5/16/2008 12:10



Max. Depth: 47.080 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

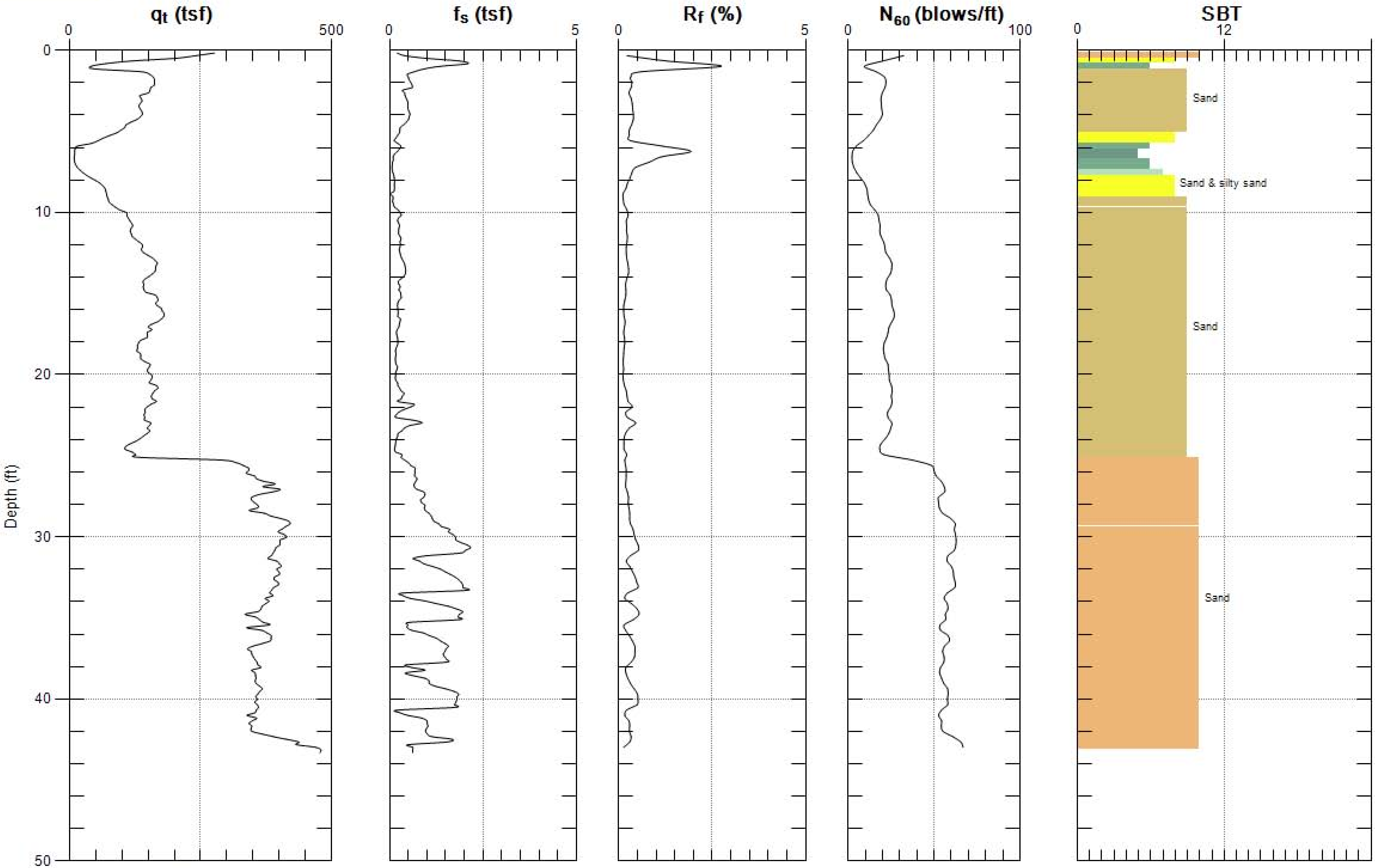
TERRA COSTA

Site: MARINA PARK

Sounding: CPT-12

Engineer: B. SMILLE

Date: 5/16/2008 12:31



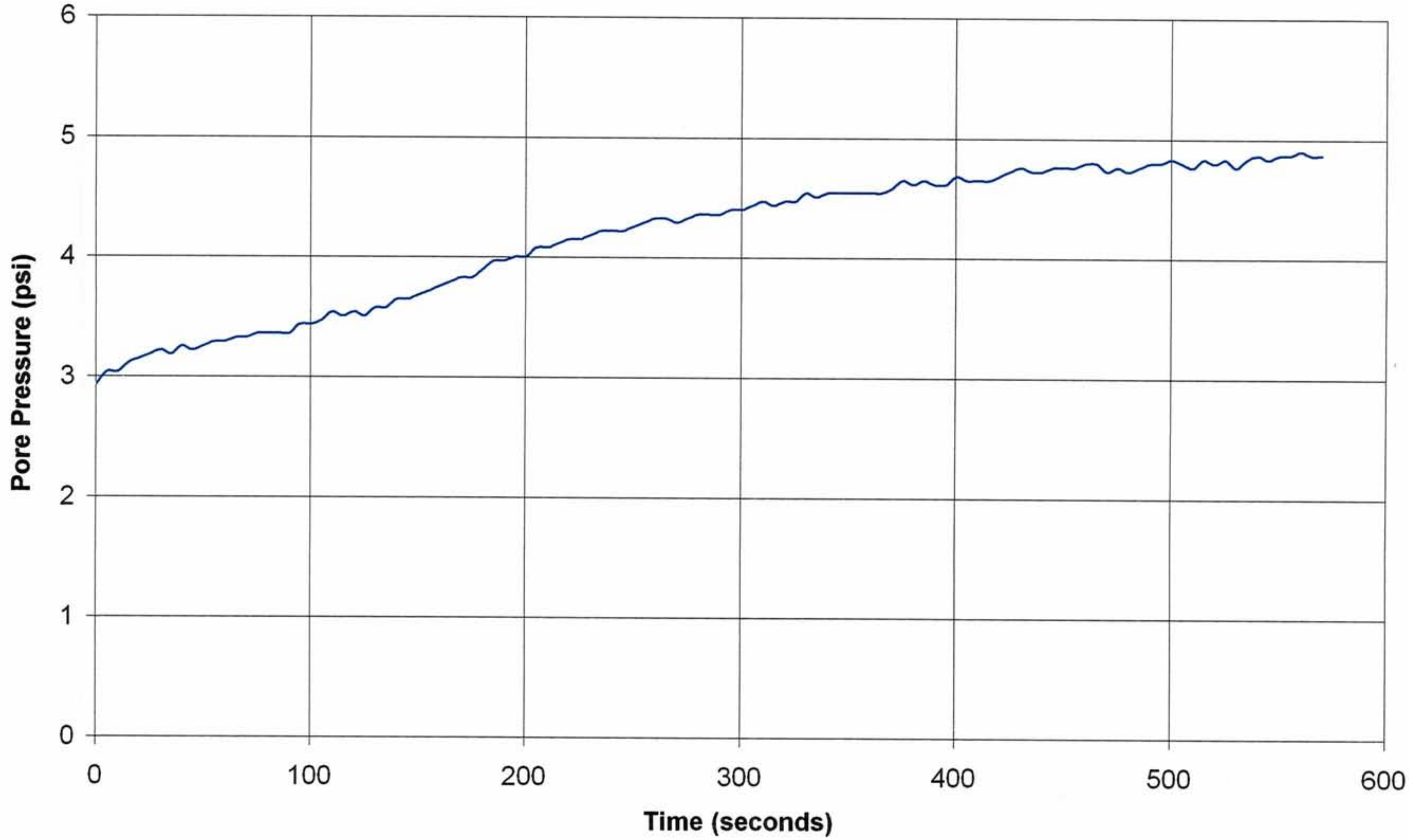
Max. Depth: 43.307 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

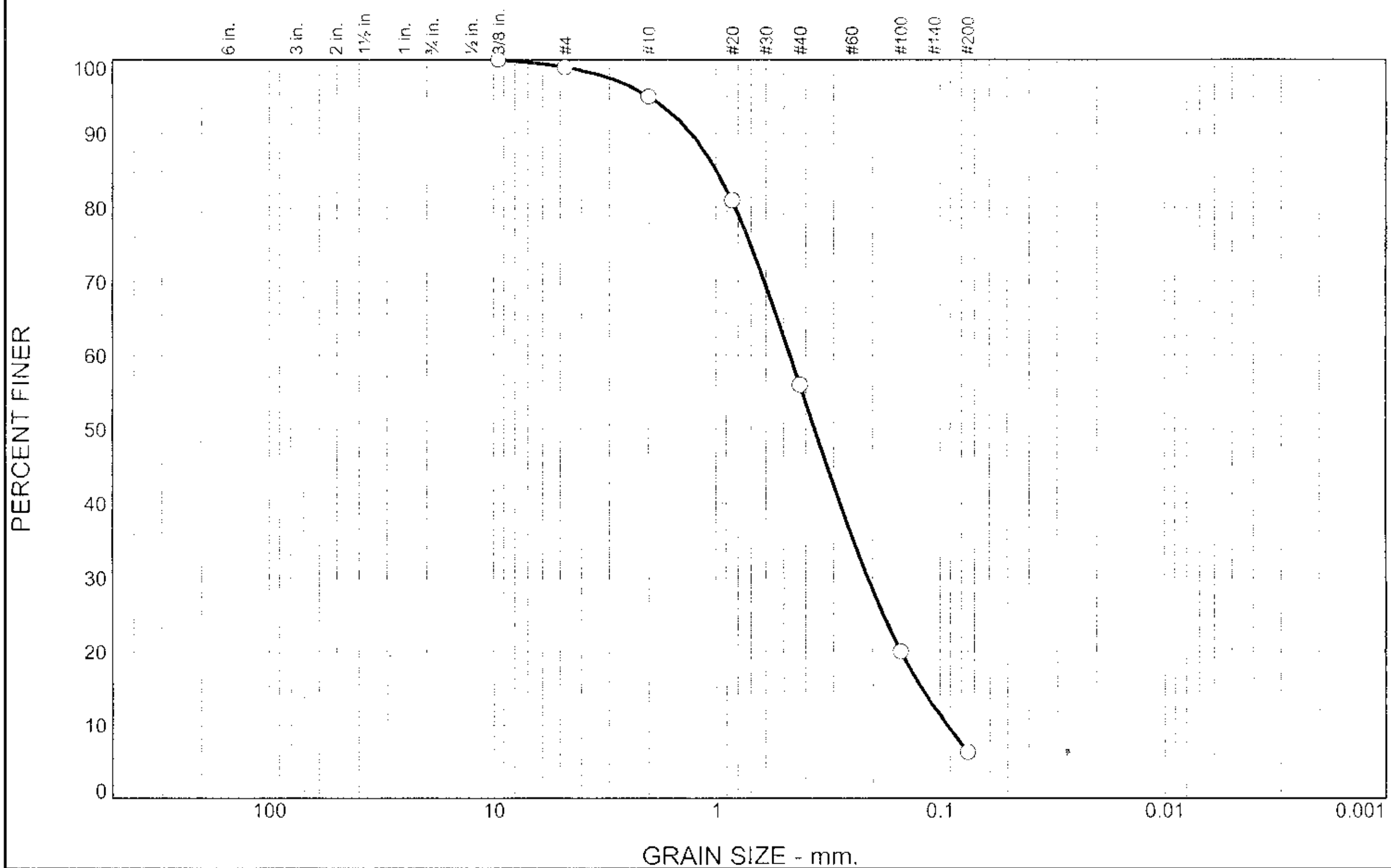
Sounding: CPT-10
Depth: 22.146
Site: MARINA PARK
Engineer: G. SPAULDING



APPENDIX B

LABORATORY TEST RESULTS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.0	4.0	39.0	49.7	6.3	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375"	100.0		
#4	99.0		
#10	95.0		
#20	81.0		
#40	56.0		
#100	20.0		
#200	6.3		

* (no specification provided)

Material Description

(Lab #19844)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.9950 D_{60}= 0.4701 D_{50}= 0.3650
D_{30}= 0.2109 D_{15}= 0.1208 D_{10}= 0.0930
C_u= 5.05 C_c= 1.02

Classification

USCS= AASHTO=

Remarks

As received moisture content=15.9%

Sample Number: B1-1 **Depth:** 5' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19844

Tested By: Valles/Stacy **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 5'
Sample Number: B1-1
Material Description: (Lab #19844)
Date: 5/29/08
Testing Remarks: As received moisture content=15.9%
Tested by: Valles/Stacy
Checked by: Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.375"	100.0
#4	99.0
#10	95.0
#20	81.0
#40	56.0
#100	20.0
#200	6.3

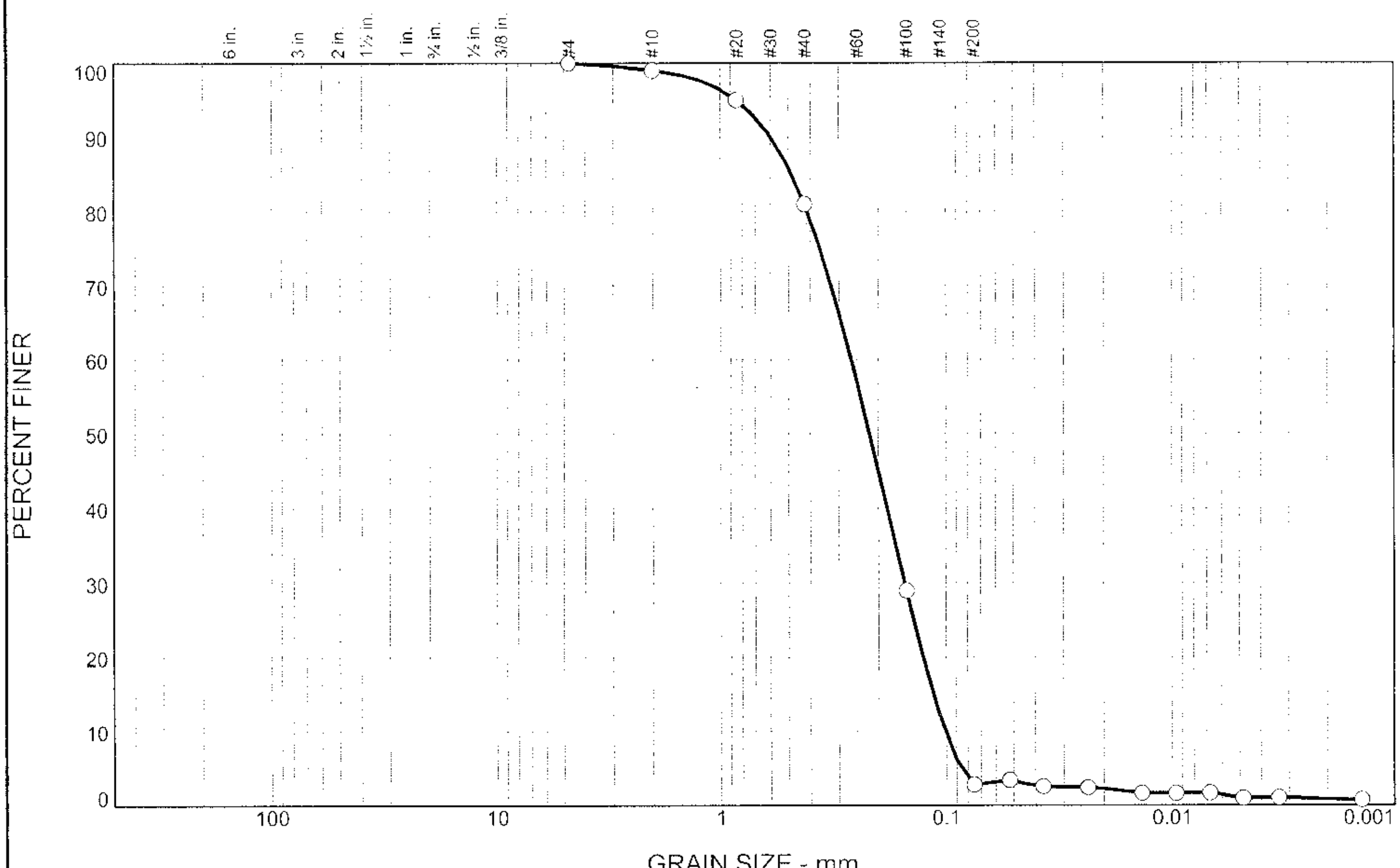
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	1.0	1.0	4.0	39.0	49.7	92.7			6.3

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.0930	0.1208	0.1500	0.2109	0.3650	0.4701	0.8209	0.9950	1.2932	2.0000

Fineness Modulus	C_u	C_c
1.84	5.05	1.02

Particle Size Distribution Report



% +3"	% Gravel Coarse	% Gravel Fine	% Sand Coarse	% Sand Medium	% Sand Fine	% Fines Silt	% Fines Clay
0.0	0.0	0.0	1.0	18.0	78.2	1.8	1.0

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.0		
#20	95.0		
#40	81.0		
#100	29.0		
#200	2.8		

* (no specification provided)

Material Description

(Lab #19845)

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.4834 D_{60}= 0.2643 D_{50}= 0.2192
D_{30}= 0.1528 D_{15}= 0.1137 D_{10}= 0.1008
C_u= 2.62 C_c= 0.88

Classification

USCS= SP AASHTO=

Remarks

As received moisture content=24.6%

Sample Number: B1-2 **Depth:** 10' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19845

Tested By: Valles **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 10' **Sample Number:** B1-2
Material Description: (Lab #19845)
Date: 5/29/08
USCS Classification: SP
Testing Remarks: As received moisture content=24.6%
Tested by: Valles **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
#4	100.0
#10	99.0
#20	95.0
#40	81.0
#100	29.0
#200	2.8

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 99.0
Weight of hydrometer sample =116.88
Hygroscopic moisture correction:
Moist weight and tare = 33.95
Dry weight and tare = 33.92
Tare weight = 20.67
Hygroscopic moisture = 0.2%
Table of composite correction values:

Temp., deg. C:	18.0	19.8	21.6	27.7
Comp. corr.:	-8.0	-7.0	-6.0	-5.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.65
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 **x Rm**

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	19.8	11.0	4.0	0.0137	11.0	14.5	0.0521	3.4
2.00	19.8	10.0	3.0	0.0137	10.0	14.7	0.0370	2.5
5.00	19.5	10.0	2.8	0.0137	10.0	14.7	0.0235	2.4
15.00	19.7	9.0	1.9	0.0137	9.0	14.8	0.0136	1.7
30.00	19.7	9.0	1.9	0.0137	9.0	14.8	0.0096	1.7
60.00	19.8	9.0	2.0	0.0137	9.0	14.8	0.0068	1.7
120.00	20.0	8.0	1.1	0.0136	8.0	15.0	0.0048	0.9
250.00	20.2	8.0	1.2	0.0136	8.0	15.0	0.0033	1.0
1440.00	19.6	8.0	0.9	0.0137	8.0	15.0	0.0014	0.8

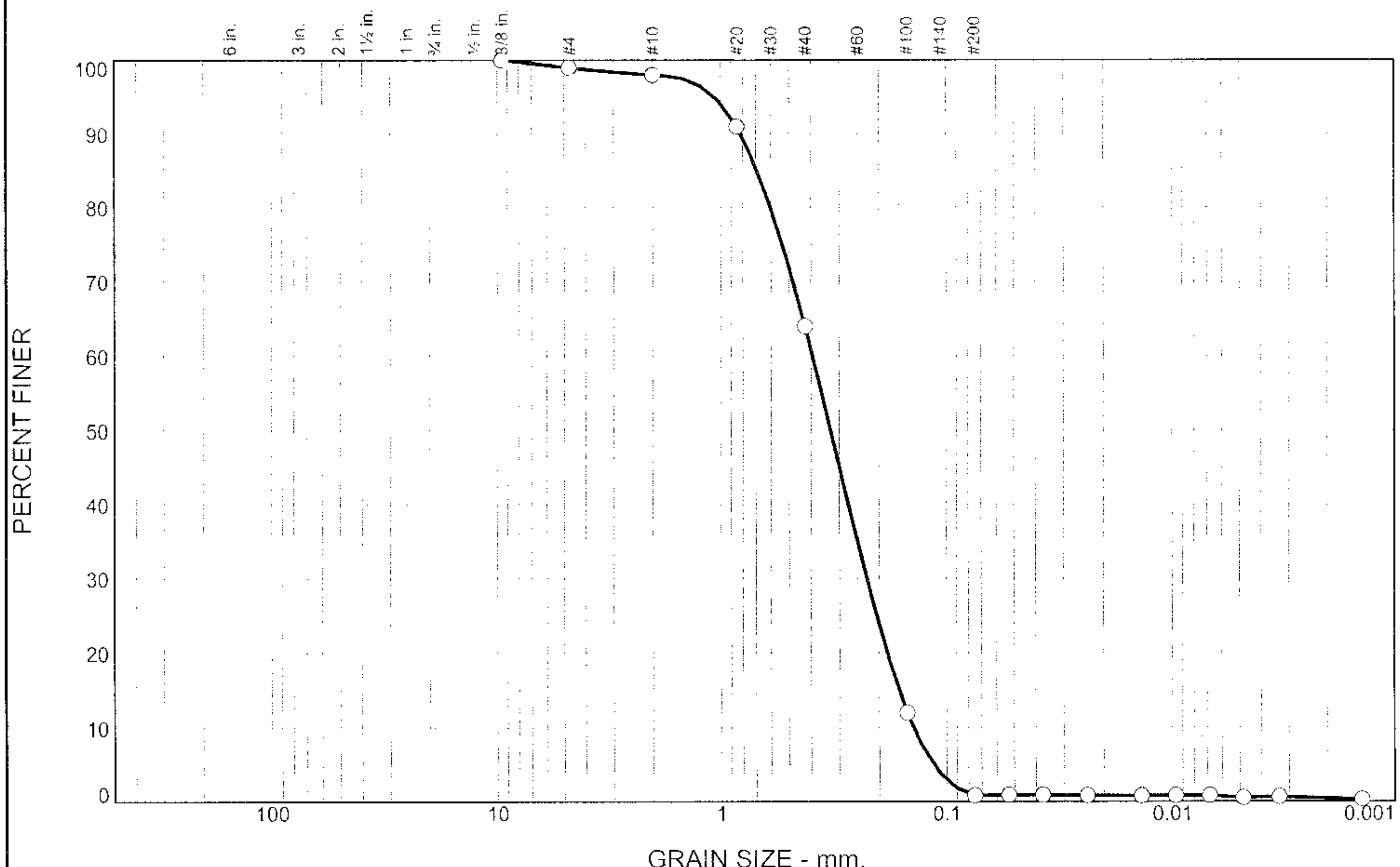
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	1.0	18.0	78.2	97.2	1.8	1.0	2.8

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1008	0.1137	0.1264	0.1528	0.2192	0.2643	0.4130	0.4834	0.5980	0.8500

Fineness Modulus	C_u	C_c
1.18	2.62	0.88

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.0	1.0	34.0	63.2	0.3	0.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375"	100.0		
#4	99.0		
#10	98.0		
#20	91.0		
#40	64.0		
#100	12.0		
#200	0.8		

* (no specification provided)

Material Description

(Lab #19846)

Atterberg Limits

PL= NV LL= PI= NP

Coefficients

D_{85}= 0.6900 D_{60}= 0.3937 D_{50}= 0.3276
D_{30}= 0.2267 D_{15}= 0.1631 D_{10}= 0.1408
C_u= 2.80 C_c= 0.93

Classification

USCS= SP AASHTO=

Remarks

As received moisture content=19.7%

Sample Number: B1-3 **Depth:** 15' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19846

Tested By: Valles **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 15' **Sample Number:** B1-3
Material Description: (Lab #19846)
Date: 5/29/08 **PL:** NV **PI:** NP
USCS Classification: SP
Testing Remarks: As received moisture content=19.7%
Tested by: Valles **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.375"	100.0
#4	99.0
#10	98.0
#20	91.0
#40	64.0
#100	12.0
#200	0.8

Hydrometer Test Data

Hydrometer test uses material passing #10
Percent passing #10 based upon complete sample = 98.0
Weight of hydrometer sample =117.61
Hygroscopic moisture correction:
Moist weight and tare = 33.08
Dry weight and tare = 33.04
Tare weight = 20.68
Hygroscopic moisture = 0.3%
Table of composite correction values:

Temp., deg. C:	18.0	19.8	21.6	27.7
Comp. corr.:	-8.0	-7.0	-6.0	-5.0

Meniscus correction only = 0.0
Specific gravity of solids = 2.65
Hydrometer type = 152H
Hydrometer effective depth equation: L = 16.294964 - 0.164 x Rm

Elapsed Time (min.)	Temp. (deg. C.)	Actual Reading	Corrected Reading	K	Rm	Eff. Depth	Diameter (mm.)	Percent Finer
1.00	19.9	8.0	1.1	0.0137	8.0	15.0	0.0529	0.9
2.00	19.9	8.0	1.1	0.0137	8.0	15.0	0.0374	0.9
5.00	19.8	8.0	1.0	0.0137	8.0	15.0	0.0237	0.8
15.00	19.7	8.0	0.9	0.0137	8.0	15.0	0.0137	0.8
30.00	19.8	8.0	1.0	0.0137	8.0	15.0	0.0097	0.8
60.00	19.8	8.0	1.0	0.0137	8.0	15.0	0.0068	0.8
120.00	20.0	7.5	0.6	0.0136	7.5	15.1	0.0048	0.5
250.00	20.3	7.5	0.8	0.0136	7.5	15.1	0.0033	0.7
1440.00	19.6	7.5	0.4	0.0137	7.5	15.1	0.0014	0.3

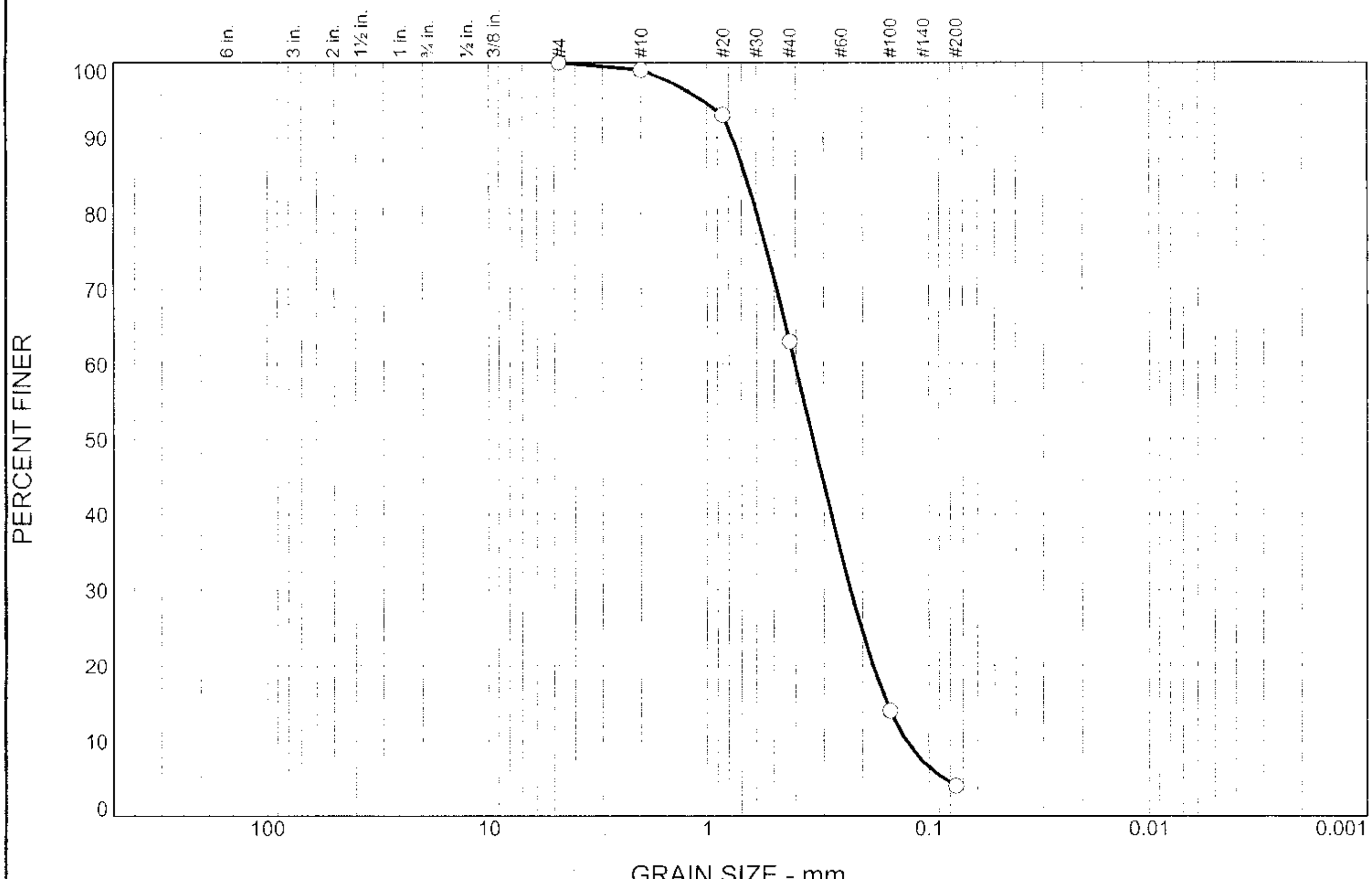
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	1.0	1.0	1.0	34.0	63.2	98.2	0.3	0.5	0.8

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1408	0.1631	0.1841	0.2267	0.3276	0.3937	0.6023	0.6900	0.8160	1.0624

Fineness Modulus	C_u	C_c
1.70	2.80	0.93

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.0	36.0	59.0	4.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.0		
#20	93.0		
#40	63.0		
#100	14.0		
#200	4.0		

* (no specification provided)

Material Description

(Lab #19847)

Atterberg Limits

PL= NV LL= PI= NP

Coefficients

D_{85}= 0.6693 D_{60}= 0.4020 D_{50}= 0.3346
D_{30}= 0.2270 D_{15}= 0.1552 D_{10}= 0.1266
C_u= 3.18 C_c= 1.01

Classification

USCS= SP AASHTO=

Remarks

As received moisture content=20.0%

Sample Number: B1-4 **Depth:** 20' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19847

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 20' **Sample Number:** B1-4
Material Description: (Lab #19847)
Date: 5/29/08 **PL:** NV **PI:** NP
USCS Classification: SP
Testing Remarks: As received moisture content=20.0%

Sieve Test Data

Sieve Opening Size	Percent Finer
#4	100.0
#10	99.0
#20	93.0
#40	63.0
#100	14.0
#200	4.0

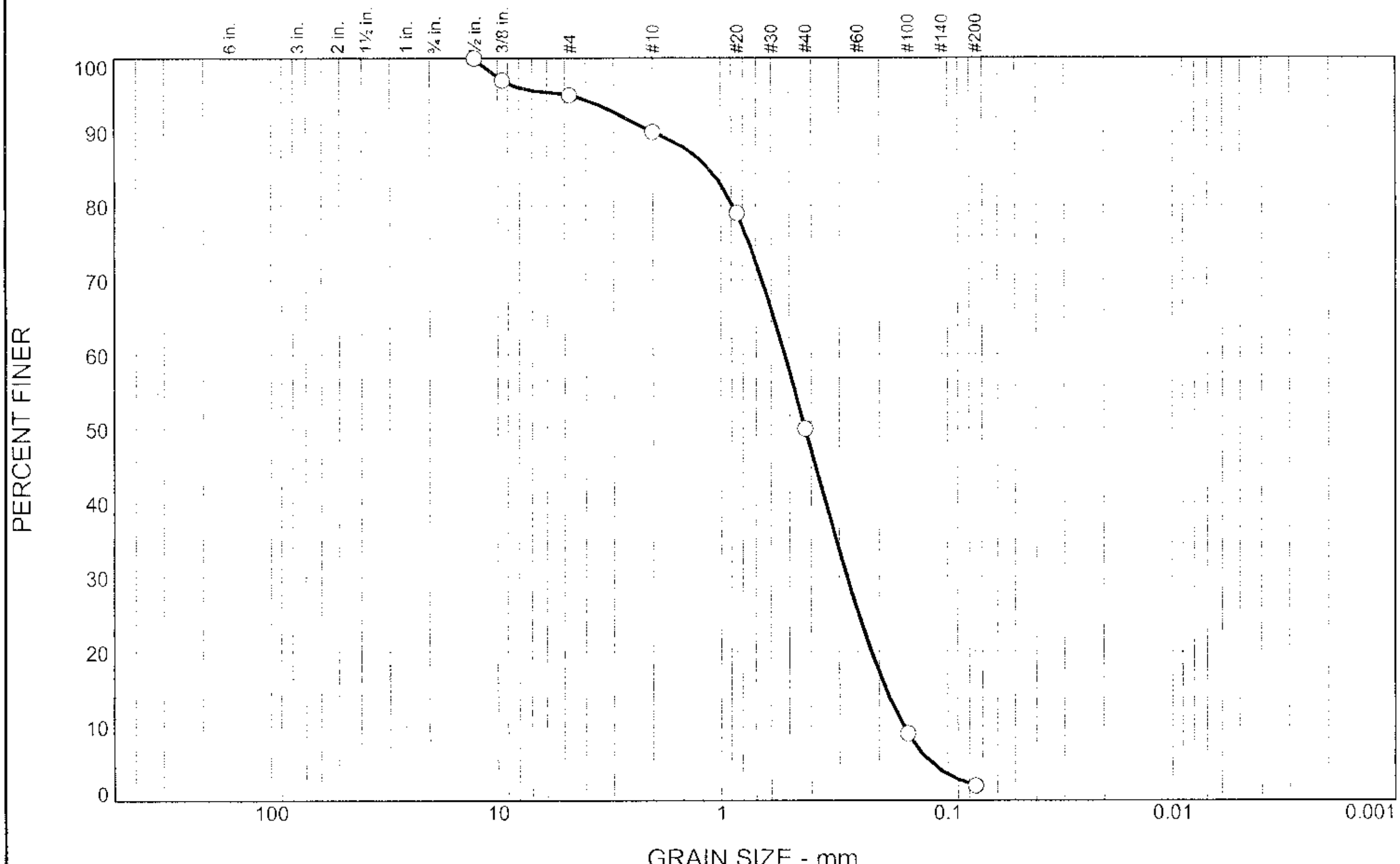
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	1.0	36.0	59.0	96.0			4.0

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1266	0.1552	0.1797	0.2270	0.3346	0.4020	0.5956	0.6693	0.7674	1.0568

Fineness Modulus	C_u	C_c
1.66	3.18	1.01

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.0	5.0	40.0	48.0	2.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.5"	100.0		
0.375"	97.0		
#4	95.0		
#10	90.0		
#20	79.0		
#40	50.0		
#100	9.0		
#200	2.0		

* (no specification provided)

Material Description

SP (Lab #19848)

Atterberg Limits

PL= NV LL= PI= NP

Coefficients

D_{85}= 1.1243 D_{60}= 0.5239 D_{50}= 0.4250
D_{30}= 0.2771 D_{15}= 0.1874 D_{10}= 0.1567
C_u= 3.34 C_c= 0.94

Classification

USCS= SP AASHTO=

Remarks

As reveived moisture content=18.5%

Sample Number: B1-5 **Depth:** 25' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19848

Tested By: Sancha/Stacy **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 25' **Sample Number:** B1-5
Material Description: SP (Lab #19848)
Date: 5/29/08 **PL:** NV **PI:** NP
USCS Classification: SP
Testing Remarks: As revcived moisture content=18.5%
Tested by: Sancha/Stacy **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.5"	100.0
0.375"	97.0
#4	95.0
#10	90.0
#20	79.0
#40	50.0
#100	9.0
#200	2.0

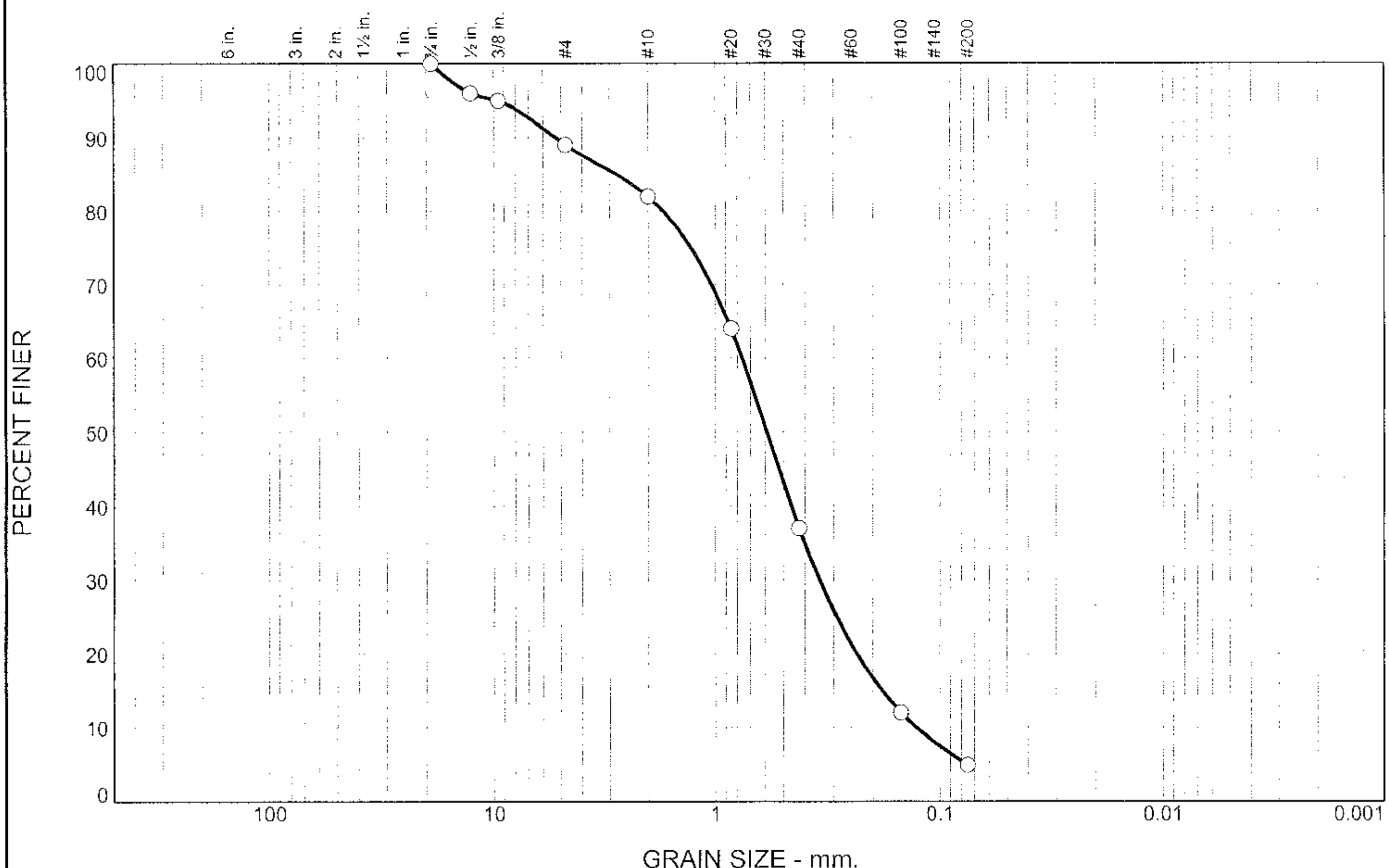
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	5.0	5.0	5.0	40.0	48.0	93.0			2.0

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1567	0.1874	0.2166	0.2771	0.4250	0.5239	0.8814	1.1243	2.0000	4.7500

Fineness Modulus	C_u	C_c
2.23	3.34	0.94

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	11.0	7.0	45.0	32.1	4.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	96.0		
0.375"	95.0		
#4	89.0		
#10	82.0		
#20	64.0		
#40	37.0		
#100	12.0		
#200	4.9		

* (no specification provided)

Material Description

SP (Lab #19849)

Atterberg Limits

PL= NV LL= PI= NP

Coefficients

D_{85}= 2.7828 D_{60}= 0.7598 D_{50}= 0.5902
D_{30}= 0.3453 D_{15}= 0.1824 D_{10}= 0.1281
C_u= 5.93 C_c= 1.23

Classification

USCS= SP AASHTO=

Remarks

As received moisture content=11.1%

Sample Number: B2-1 **Depth:** 5' **Date:** 5/30/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19849

Tested By: Valles/Stacy **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

6/16/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 5' **Sample Number:** B2-1
Material Description: SP (Lab #19849)
Date: 5/30/08 **PL:** NV **PI:** NP
USCS Classification: SP
Testing Remarks: As received moisture content=11.1%
Tested by: Valles/Stacy **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.75"	100.0
0.5"	96.0
0.375"	95.0
#4	89.0
#10	82.0
#20	64.0
#40	37.0
#100	12.0
#200	4.9

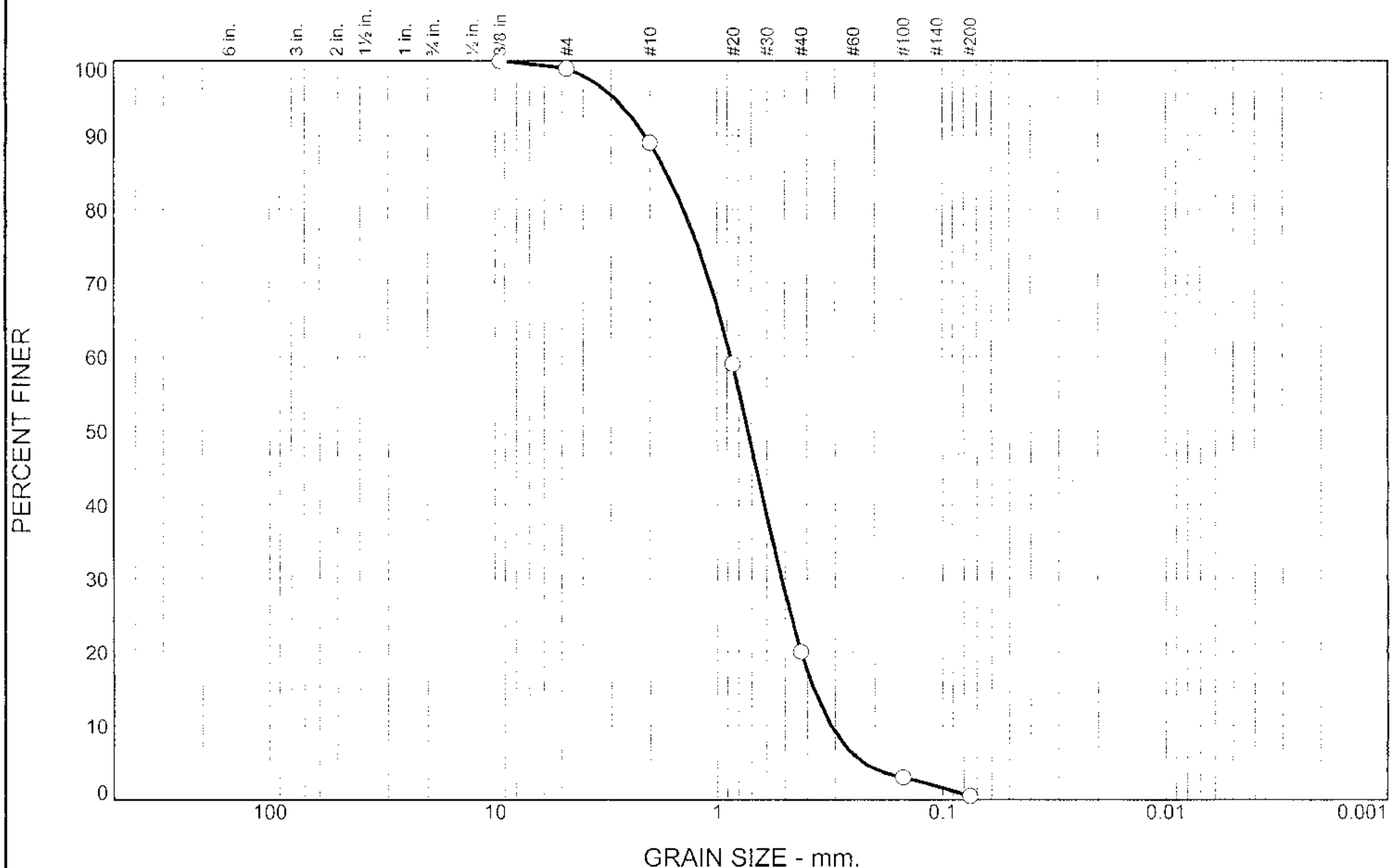
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	11.0	11.0	7.0	45.0	32.1	84.1			4.9

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1281	0.1824	0.2360	0.3453	0.5902	0.7598	1.7060	2.7828	5.2834	9.5250

Fineness Modulus	C_u	C_c
2.71	5.93	1.23

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.0	10.0	69.0	19.5	0.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375"	100.0		
#4	99.0		
#10	89.0		
#20	59.0		
#40	20.0		
#100	3.0		
#200	0.5		

* (no specification provided)

Material Description

SP (Lab #19850)

Atterberg Limits

PL= NV LL= PI= NP

Coefficients

D_{85}= 1.6791 D_{60}= 0.8662 D_{50}= 0.7255
D_{30}= 0.5181 D_{15}= 0.3737 D_{10}= 0.3133
C_u= 2.77 C_c= 0.99

Classification

USCS= SP AASHTO=

Remarks

As received moisture content=19.0%

Sample Number: B2-2 **Depth:** 10-11' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19850

Tested By: Sancha/Stacy **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 10-11' **Sample Number:** B2-2
Material Description: SP (Lab #19850)
Date: 5/29/08 **PL:** NV **PI:** NP
USCS Classification: SP
Testing Remarks: As received moisture content=19.0%
Tested by: Sancha/Stacy **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.375"	100.0
#4	99.0
#10	89.0
#20	59.0
#40	20.0
#100	3.0
#200	0.5

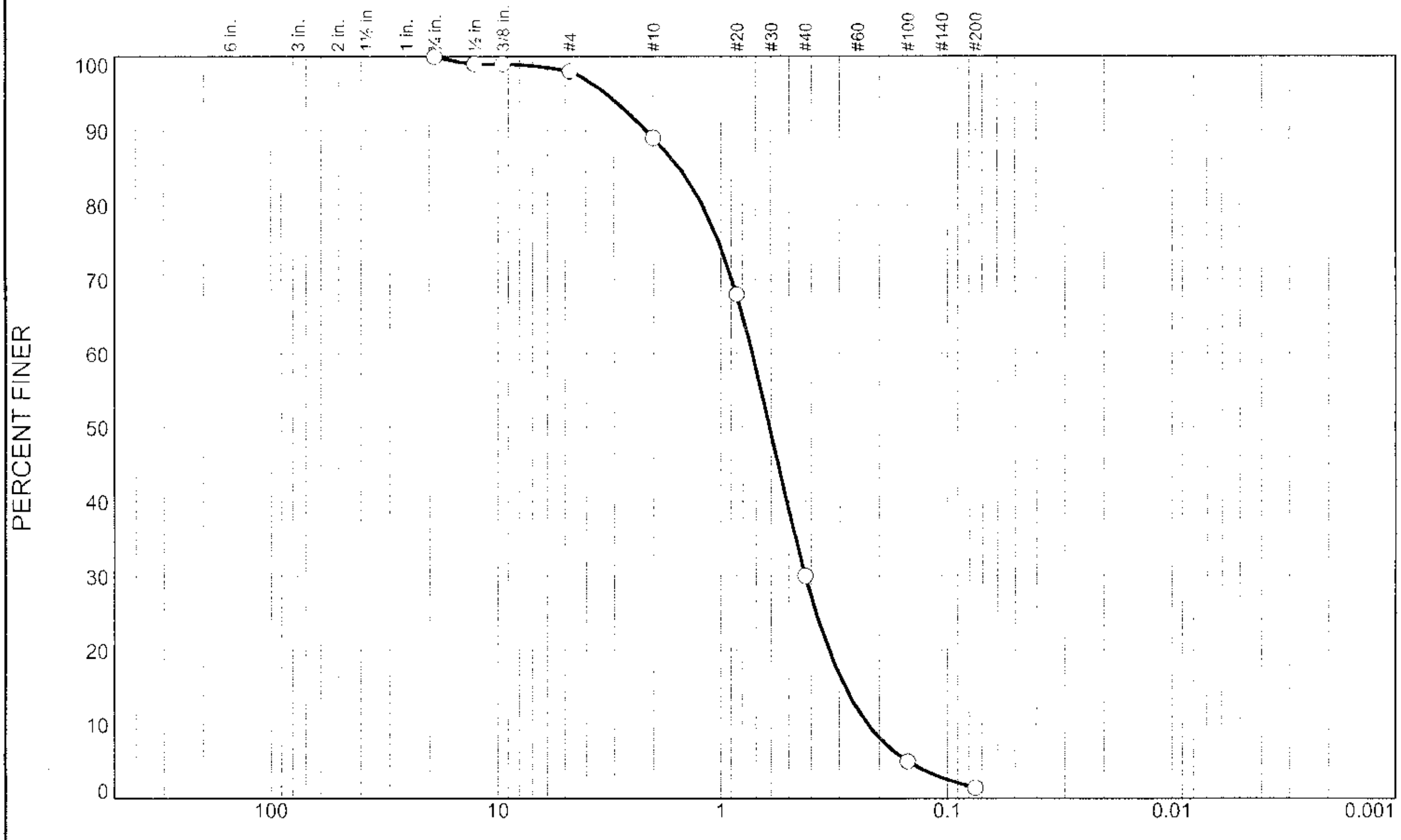
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	1.0	1.0	10.0	69.0	19.5	98.5			0.5

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.3133	0.3737	0.4250	0.5181	0.7255	0.8662	1.4066	1.6791	2.1030	2.8913

Fineness Modulus	C_u	C_c
2.84	2.77	0.99

Particle Size Distribution Report



% +3"	% Gravel Coarse	% Gravel Fine	% Sand Coarse	% Sand Medium	% Sand Fine	% Fines Silt	% Fines Clay
0.0	0.0	2.0	9.0	59.0	28.6	1.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75"	100.0		
0.5"	99.0		
0.375"	99.0		
#4	98.0		
#10	89.0		
#20	68.0		
#40	30.0		
#100	5.0		
#200	1.4		

* (no specification provided)

Material Description

SP (Lab #19851)

Atterberg Limits

PL= NV LL= PI= NP

Coefficients

D_{85}= 1.5276 D_{60}= 0.7253 D_{50}= 0.6087
D_{30}= 0.4250 D_{15}= 0.2827 D_{10}= 0.2250
C_u= 3.22 C_c= 1.11

Classification

USCS= SP AASHTO=

Remarks

As received moisture content=16.6%

Sample Number: B2-4 **Depth:** 20' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19851

Tested By: Stacy/Valles **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 20' **Sample Number:** B2-4
Material Description: SP (Lab #19851)
Date: 5/29/08 **PL:** NV **PI:** NP
USCS Classification: SP
Testing Remarks: As received moisture content=16.6%
Tested by: Stacy/Valles **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
0.75"	100.0
0.5"	99.0
0.375"	99.0
#4	98.0
#10	89.0
#20	68.0
#40	30.0
#100	5.0
#200	1.4

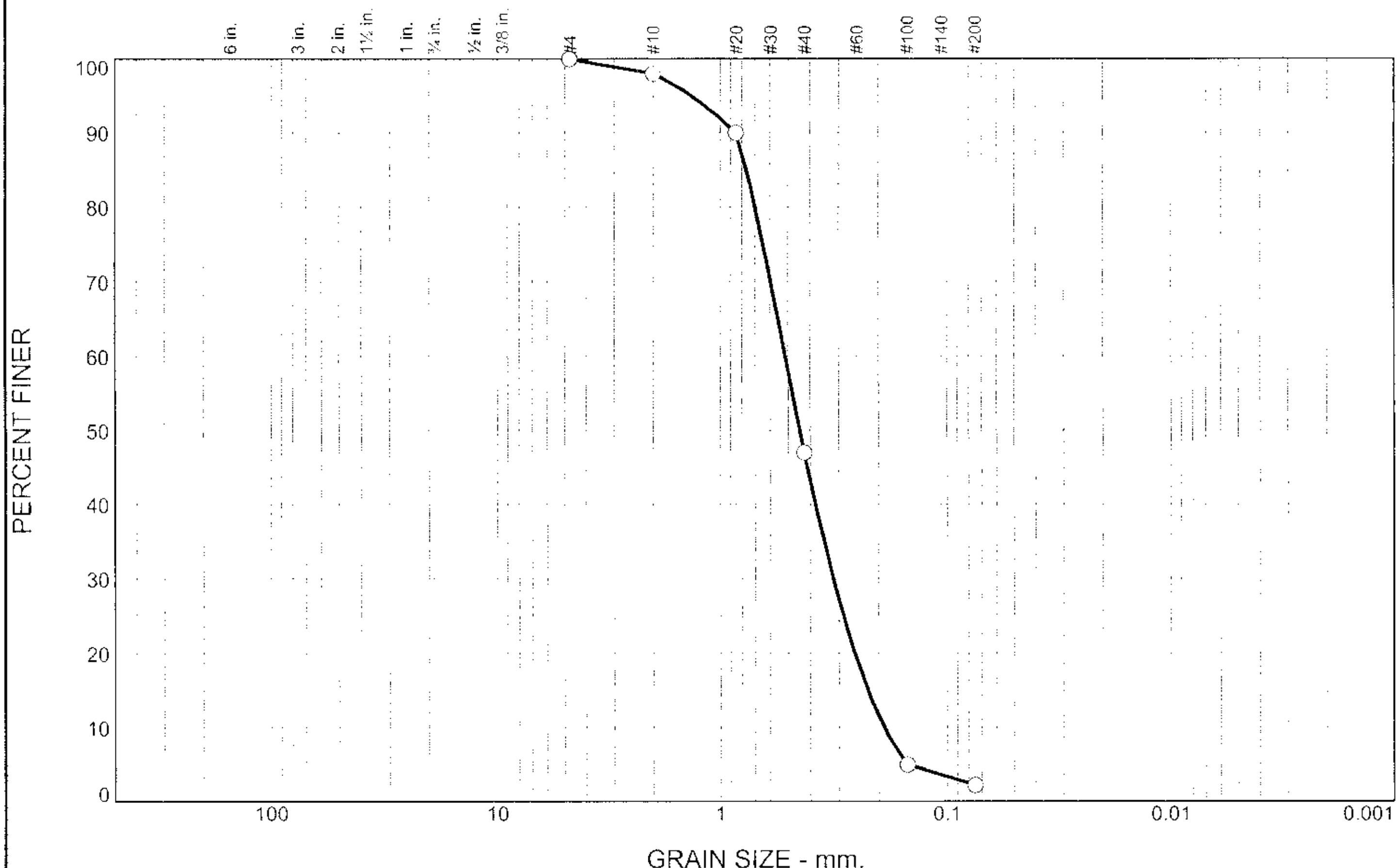
Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	2.0	2.0	9.0	59.0	28.6	96.6			1.4

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.2250	0.2827	0.3334	0.4250	0.6087	0.7253	1.2022	1.5276	2.1549	3.2640

Fineness Modulus	C_u	C_c
2.61	3.22	1.11

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	2.0	51.0	44.8	2.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	98.0		
#20	90.0		
#40	47.0		
#100	5.0		
#200	2.2		

* (no specification provided)

Material Description

SP (Lab #19852)

Atterberg Limits

PL= NV LL= PI= NP

Coefficients

D_{85}= 0.7628 D_{60}= 0.5147 D_{50}= 0.4448
D_{30}= 0.3171 D_{15}= 0.2237 D_{10}= 0.1901
C_u= 2.71 C_c= 1.03

Classification

USCS= SP AASHTO=

Remarks

As received moisture content=19.8%

Sample Number: B2-6 **Depth:** 30' **Date:** 5/29/08

MACTEC, Inc.

San Diego, California

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project No: 5014-07-0012.25 **Figure** #19852

Tested By: Stacy/Sancha **Checked By:** Collins

GRAIN SIZE DISTRIBUTION TEST DATA

5/30/2008

Client: TerraCosta Consulting Group, Inc.
Project: #2573 Marina Park
Project Number: 5014-07-0012.25
Depth: 30' **Sample Number:** B2-6
Material Description: SP (Lab #19852)
Date: 5/29/08 **PL:** NV **PI:** NP
USCS Classification: SP
Testing Remarks: As received moisture content=19.8%
Tested by: Stacy/Sancha **Checked by:** Collins

Sieve Test Data

Sieve Opening Size	Percent Finer
#4	100.0
#10	98.0
#20	90.0
#40	47.0
#100	5.0
#200	2.2

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.0	0.0	0.0	2.0	51.0	44.8	97.8			2.2

D_{10}	D_{15}	D_{20}	D_{30}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
0.1901	0.2237	0.2553	0.3171	0.4448	0.5147	0.6970	0.7628	0.8500	1.3398

Fineness Modulus	C_u	C_c
2.05	2.71	1.03

L A B O R A T O R Y   R E P O R T

Telephone (619) 425-1993   Fax 425-7917   Established 1928

C L A R K S O N   L A B O R A T O R Y   A N D   S U P P L Y   I N C.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
A N A L Y T I C A L   A N D   C O N S U L T I N G   C H E M I S T S

Date: August 7, 2008
Purchase Order Number: 2573
Sales Order Number: 93846
Account Number: TERC

To:
--
Terra Costa Consulting Group
4455 Murphy Canyon Road, Suite 100
San Diego, Ca 92123
Attention: Gregory Spaulding

Laboratory Number: SO3412   Customers Phone: 858-573-6900
Fax: 858-573-8900

Sample Designation:
--
One soil sample received on 08/07/08, taken on 08/07/08
from Marina Park Project# 2573 marked as HA-1 @ 2-4'.

Analysis By California Test 643, 1993, Department of Transportation
Division of Construction, Method for Estimating the Service Life of
Steel Culverts.

pH 7.0

Water Added (ml)	Resistivity (ohm-cm)
10	49000
5	35000
5	24000
5	18000
5	14000
5	12000
5	11000
5	13000
5	15000

40 years to perforation for a 16 gauge metal culvert.
52 years to perforation for a 14 gauge metal culvert.
72 years to perforation for a 12 gauge metal culvert.
93 years to perforation for a 10 gauge metal culvert.
113 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.002%
Water Soluble Chloride	Calif. Test 422	0.002%

Laura Torres

Laura Torres
LT/ram

APPENDIX C

SUGGESTED ITEMS FOR INCLUSION IN SPECIFICATIONS FOR PILE DRIVING

APPENDIX C

SUGGESTED ITEMS FOR INCLUSION IN SPECIFICATIONS FOR PILE DRIVING

1.0 SCOPE

Furnish and install piling, complete, as shown and specified.

2.0 GENERAL

A. Code Requirements - Per (Uniform Building Code) (Standard Specifications for Public Works Construction), and other applicable regulations; strictest requirements govern.

B. Qualification - Piling subcontractor shall be qualified and experienced in this work. He shall present to Owner evidence of past successful installations of similar types of projects.

C. Responsibility - Owner shall accept no responsibility for the driveability of piles as shown and specified.

D. Grading - Necessary clearing, excavating, and filling shall be done by the General Contractor.

E. Pile Locations - Staked out pile locations shall be protected from damage or movement. Cost for replacing moved or damaged stakes shall be borne by the Contractor under this section of work.

F. Available Data - Records of the borings made at this work site are available at the Owner's office. These records pertain to conditions at the boring locations. Contractors are expected to make a personal inspection of the site and to otherwise satisfy themselves as to the conditions affecting the work. No claims for extra compensation or extension of time shall be allowed on account of subsurface conditions inconsistent with the data given.

G. Pile Depth - All piles shall be advanced to the tip elevations shown on the plans. Piles stopped at lesser depths shall be cause for rejection. (See Section 5.0, Installation).

H. Inspection - The Owner's representative shall inspect the placement of all piles. At least one week's notice shall be given before the first pile is driven.

3.0 MATERIALS

Concrete Piles

A. Concrete - Minimum 28-day compressive strength: (5,000) psi.

B. Prestressing Strand - ASTM-(A416), uncoated (7) wire cold drawn type; ultimate stress (250,000) psi.

C. Mild Reinforcing - ASTM-(A15), intermediate grade.

D. Wire for Special Reinforcing - ASTM-(A82), cold drawn wire.

Steel Sheet Piles

A. Steel sheet piles shall conform to normal material specifications: ASTM A328, ASTM A572 Grades 42 through 55.

4.0 HANDLING OF PILES

All piles shall be handled with care to avoid damage. Damage to any pile prior to driving shall be cause for immediate rejection.

5.0 INSTALLATION

A. General - Drive the first four piles at selected locations shown to the tip elevations shown on the plans. The indicator piles shall be driven with the same size and type of hammer to be used for driving the production piles. Indicator piles will be selected from permanent piles. Driving criteria will be established during construction by the Geotechnical Engineer on the basis of the first piles before additional piles are driven. Each pile shall be marked at one-foot intervals along its length to facilitate recording of penetration resistance. Drive each pile without interruption, until design depth is attained. If unforeseen causes arise, only by written permission shall deviation from this procedure be allowed. Refusal driving criteria will be determined by the Geotechnical Engineer during construction.

All piles shall be placed at the locations specified on the contract drawings.

B. Record of Driving - Kept by Piling Inspector selected and paid for by Owner.

1. Reference - All piles per numbering system.

2. Dimensions - Include elevation of tip and butt before and after cutting off.

3. Driving Resistance - Complete record with number of blows required to drive each foot for full length of each pile.

4. <u>Time</u> - Include time of starting, completion, interruptions (if any), and condition of pile after driving.

5. <u>At Completion of Work</u> - Contractor shall furnish accurate drawing showing locations of piles as driven.

C. <u>Location</u> - All piles shall be placed at the locations specified on the contract drawing. No pile shall be driven more than 3 inches in horizontal dimension from its design location.

D. <u>Alignment</u> - Do not exceed 2 percent maximum deviation from vertical over any section of length. Keep pile center at cut-off within 3 inches of design location. Pulling piles into position will not be permitted. The Contractor shall provide substitute piles where driven piles exceed specified tolerances; all correction costs shall be paid for by Contractor under this section, including any structural redesign, additional materials, and labor required for pile caps.

E. <u>Heave Checks</u> - Make on selected piles as directed by the Geotechnical Engineer. Check heave by measuring length and checking elevation on each pile immediately after it has been driven; recheck elevations and length after all adjacent piles have been driven. Redrive piles, where tips heaved more than ½ inch from original elevation. When pile heave is encountered, continue heave check and redriving until assured that pile heave does not occur.

F. <u>Damaged Piles</u>

1. <u>General</u> - Any pile driven into a previously driven pile automatically rejects both piles. Leave all pile heads sound; repair or replace damaged or defect; replace as directed with a substitute pile at no expense to the Owner. Do not drive piles damaged or suspected of damage until inspected and approved. All correction costs shall be paid for by Contractor including structural redesign, additional materials, and labor required for pile caps.

2. <u>Driving Damage</u> - Development of tension cracks, spall, or chips in the concrete within the pay length shall be cause of rejection.

G. <u>Hard Driving</u> - Difficult driving may be experienced within the stiff clays and formational sand deposits encountered above the design tip elevation of piles in the western portion of the site. All piles shall be driven to the design tip elevation unless specifically approved otherwise in writing by the Geotechnical Engineer at the time of construction.

H. Jetting is permitted for both isolated concrete piles and concrete sheets only as follows:

Jetting shall be limited to the use of internal manifolded pipes cast into the pile and shall use, to the extent practical, a low volume and low pressure water source. The proposed jet pipe configuration and pile installation procedures should be reviewed by the owner's representative prior to approval. Jetting, under approved conditions, is permitted down to within 2 feet of plan tip elevation for piles providing lateral resistance only.

Jetting is not allowed within five feet of plan tip elevation for axially-loaded piles.

I. <u>Predrilling</u> - Predrilling will be allowed for piles, but shall in no case extend to within 5 feet of the final tip elevation of any piles for support of structures. The diameter of a predrilled hole shall not exceed 10 inches. Predrilling is not recommended for piles required for uplift capacity.

J. <u>Driving Equipment</u> - Use approved type as generally used in standard pile driving practice. Use driving hammers of such size and type which are able to consistently deliver effective dynamic energy to the piles and which operate at manufacturer's recommended speeds and pressures. Pile hammer shall have a minimum rated energy of 50,000 foot-pounds per blow for 14-inch round piles.

Hammers developing greater energies or sonic hammers may be used upon written authorization of the Geotechnical Engineer. It shall be demonstrated that the proposed hammer will adequately drive the pile to the required depth without damage to the pile. Swing leads will not be permitted; use fixed leads or other suitable means for holding pile firmly in position and in alignment with the hammer. Vertical piles shall be plumb before driving. Special precautions shall be taken to insure against leading away of piles from the plumb or true position. Use suitable anvils or cushions of approved design, depending on type of pile, to prevent damage to pile. Care shall be taken during driving to prevent and correct any tendency of piles to twist, rotate, or walk.

6.0 DRIVING CRITERIA

<u>Reduction of Hammer Energy for Prestressed Piles</u> - When prestressed piles have settled into the ground under their own weight and the weight of the hammer, and the point of the pile is passing through soft soil so that there is little resistance, there is a possibility that longitudinal tensile stress will be set up in the pile shaft by the elastic shock waves traveling up and down the pile. For such driving conditions, the first hammer blows delivered to the pile shall have a lesser energy by reducing the stroke of the hammer. When the top of the pile is being driven to the final depth, the full length of the stroke and the full rated energy of the hammer shall be used to develop final driving resistance.

7.0 CLEANUP

Keep construction and storage areas free from waste material, rubbish, and debris resulting from this work.

8.0. PAYMENTS

A. General - Provide lump sum bid based on total pile length as shown based on length from cut-off to estimated pile tip elevation shown on drawings.

B. Measurement - Based on total effective length of piles in place. Effective length of individual piles measured from tip elevation to cut-off line.

C. Payment for Lineal Footage - In excess of that based upon the estimated pile tip elevation, when such excess is authorized, will be made on a unit price basis. Include such unit prices in the Bid.

D. Credit for Undriven Lineal Footage - Short of that based upon the estimated pile tip elevation will be made on a unit price basis. Include such unit price in the Bid.

9.0 SUBMITTALS BY CONTRACTOR:

A. General - For PILING, submit following in accordance with GENERAL CONDITIONS and SPECIAL CONDITIONS.

B. Prestressed Pile Design - Submit design calculations, prepared by a licensed engineer showing all pickup points and basis of design.

C. Reinforcing - Submit two copies of manufacturer's certificates of mill test reports for all reinforcing steel used.

D. Shop Drawings - Submit for approval by Structural Engineer. Show location of pickup points.

E. Guarantee - As specified.

F. Pile Driving Hammer - Submit description of proposed hammer, including manufacturer, type, model number, operating specifications, and hammer cushion, pile cushion data for review and approval by Geotechnical Engineer.

G. Load Test - Submit description of equipment and arrangement and set up of any load test for review and approval by the Geotechnical Engineer.

10.0 PILE TYPES NOT SPECIFIED

A. General - Consideration will be given to pile types other than those shown or specified. If Contractor proposes to use a type other than those shown, he shall submit to Owner for review a description of the pile and shall demonstrate by calculations and other corroborating evidence on the ability of the pile to sustain required loads. Contractor shall familiarize himself with all loading criteria.

B. Prequalification - Review proposed system with Owner and obtain written authorization before submitting proposal.

C. Engineering Design - Prepare revised foundation plans at no cost to Owner; plans to be prepared and stamped by licensed civil engineer. Comply with all local jurisdictional codes.

D. Pile Tests - If, in the opinion of the Owner, pile load tests are required to confirm the load bearing capacity, the costs of such test or tests shall be borne by Contractor.

E. Pile Caps - If the proposed alternate pile system results in increase in size and reinforcing of pile caps from those shown, said increases shall be made at no expense to the Owner.

APPENDIX D

SUMMARY CALCULATIONS

TerraCosta
Consulting Group

SHEET-PILE AND GUIDE-PILE CALCULATIONS

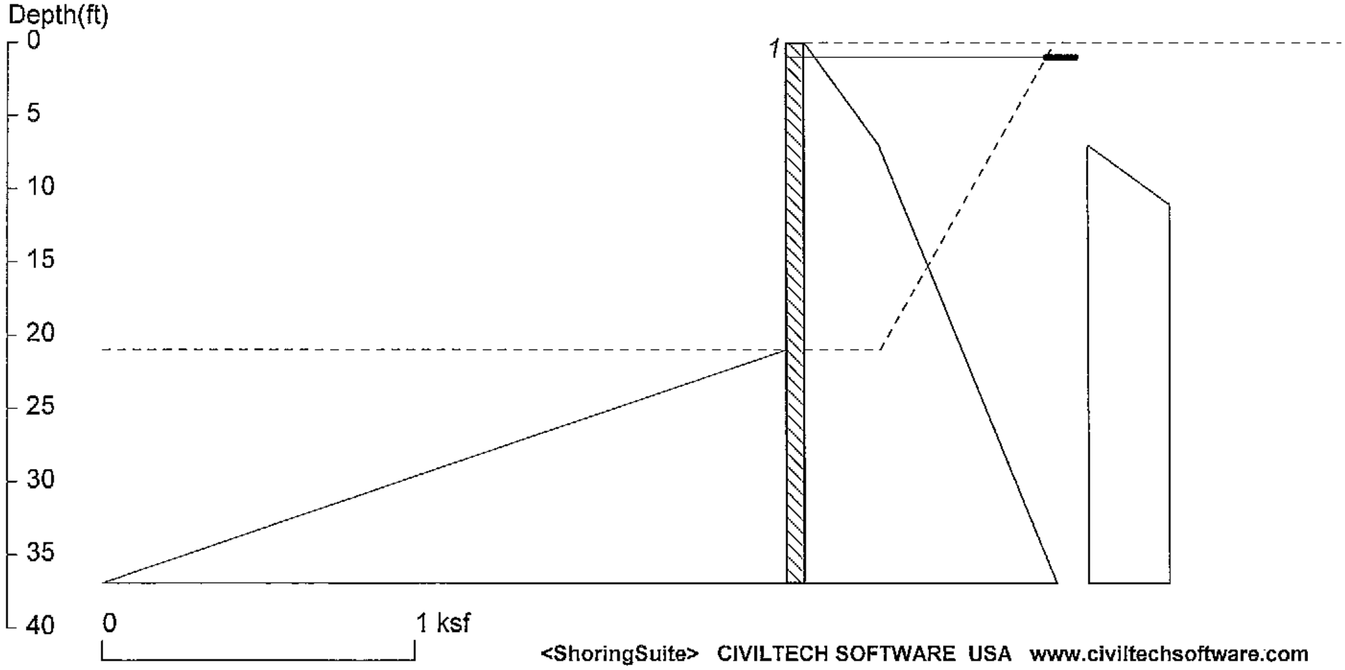
MARINA PARK PROJECT NEWPORT BEACH, CALIFORNIA

August 7, 2008

REGISTERED PROFESSIONAL ENGINEER
DAVID B. NEVIUS
No. 65015
Exp. 6/30/09
CIVIL
STATE OF CALIFORNIA

TerraCosta
Consulting Group

Marina Park +9 Seawall



Licensed to DBN TerraCosta Consulting Group
Date: 8/6/2008 File Name: UNTITLED

Wall Height=21.0 Pile Diameter=1.0 Pile Spacing=1.0

ACTIVE SPACE:	Z depth	Spacing
1	0.00	1.00
2	21.00	1.00

PASSIVE SPACE:	Z depth	Spacing
1	21.00	1.00

PILE LENGTH: Min. Embedment=15.93, Min. Pile Length=36.93
MOMENT IN PILE: Max. Moment=67.59 at Depth of 17.15

VERTICAL BEARING CAPACITY: Vertical Loading=0.0, Resistance=53.4, Vertical Factor of Safety=999.00
Request Embedment for Vertical Loading=0.0
Request Total Pile Length=21.0

PILE SELECTION:
Request Min. Section Modulus = 34.1 in3/feet, Fy= 36 ksi = 248 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
L6 (-0.06) SPZ26 (-0.17) CZ128 (-0.17) 6M (-0.05) CZ128 (-0.17)
6H (-0.05) RZ11 (-0.18) H155 (-0.19) PZ32 (-0.18) BZ20.7L (-0.17)
CZ141 (-0.16) CZ148 (-0.15) 4N (-0.14) FSPZ25 (-0.15)

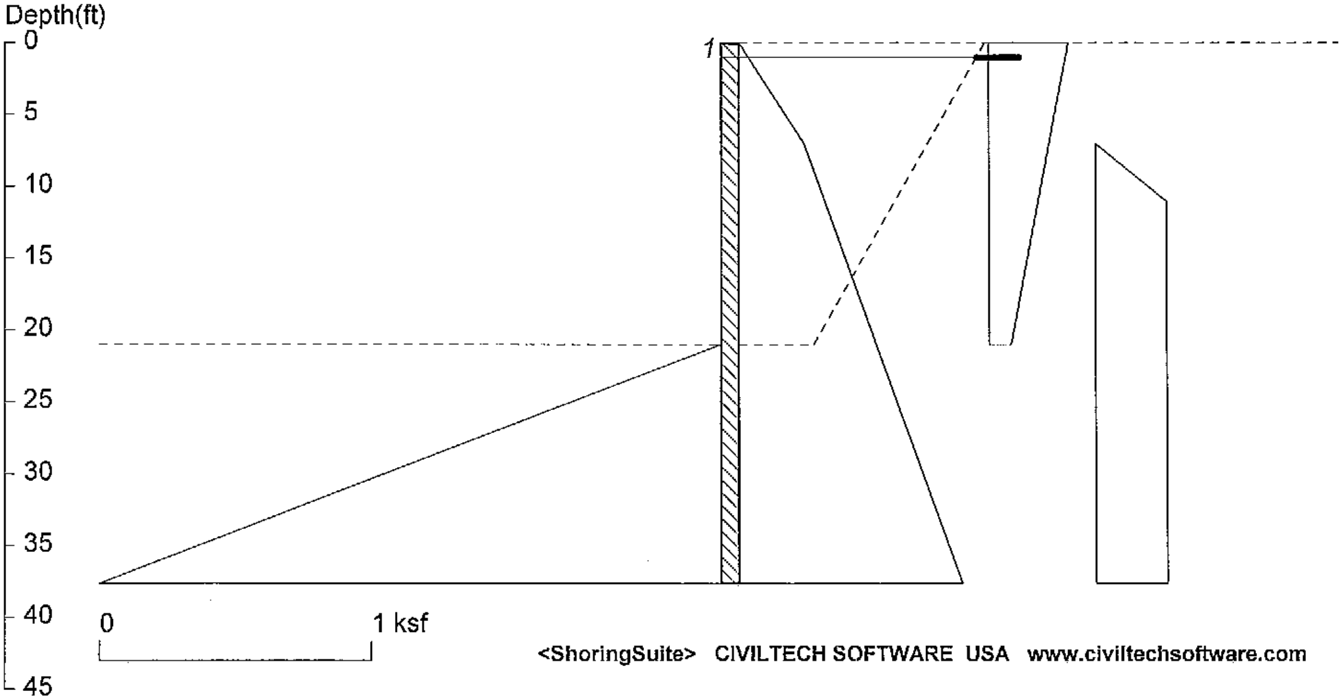
BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Total	Horiz.	Vert.	L_free	Fixed Length
1. Tieback	1.0	0.0	6.3	6.3	0.0	16.8	2.0

UNITS: Length/Depth - ft, Force - kip, Moment - kip-ft, Pressure - ksf, Pres. Slope - kip/ft3, Deflection - in

Marina Park +9 Seawall

0.30 g



Licensed to DBN TerraCosta Consulting Group
Date: 8/6/2008 File Name: UNTITLED

Wall Height=21.0 Pile Diameter=1.0 Pile Spacing=1.0

ACTIVE SPACE:	Z depth	Spacing
1	0.00	1.00
2	21.00	1.00

PASSIVE SPACE:	Z depth	Spacing
1	21.00	1.00

PILE LENGTH: Min. Embedment=16.62, Min. Pile Length=37.62
MOMENT IN PILE: Max. Moment=83.22 at Depth of 16.62

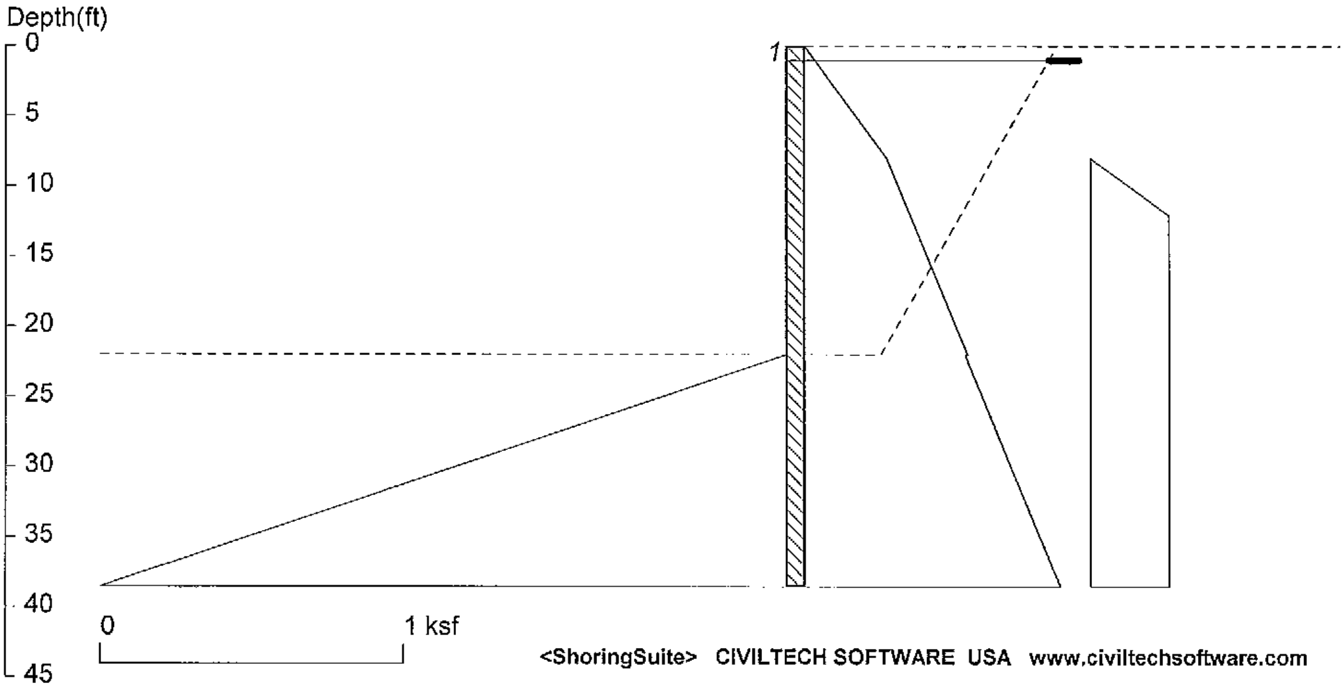
VERTICAL BEARING CAPACITY: Vertical Loading=0.0, Resistance=54.7, Vertical Factor of Safety=999.00
Request Embedment for Vertical Loading=0.0
Request Total Pile Length=21.0

PILE SELECTION:
Request Min. Section Modulus = 42.0 in3/feet, Fy= 36 ksi = 248 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
4N (-0.17) FSPZ25 (-0.18) PZ38 (-0.18) BZ26 (-0.15) AZ26 (-0.12)
H175 (-0.16) PZ35 (-0.14) H215 (-0.13) BZ32 (-0.12) FSPZ32 (-0.13)
PZ40 (-0.10) 5RU3 (-0.14) AZ36 (-0.08) BZ37 (-0.11)

BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Total	Horiz.	Vert.	L_free	Fixed Length
1. Tieback	1.0	0.0	9.4	9.4	0.0	16.8	3.0

Marina Park +10 Seawall



Licensed to DBN TerraCosta Consulting Group
Date: 8/4/2008 File Name: UNTITLED

Wall Height=22.0 Pile Diameter=1.0 Pile Spacing=1.0

ACTIVE SPACE:	Z depth	Spacing
1	0.00	1.00
2	22.00	1.00

PASSIVE SPACE:	Z depth	Spacing
1	22.00	1.00

PILE LENGTH: Min. Embedment=16.53, Min. Pile Length=38.53
MOMENT IN PILE: Max. Moment=76.23 at Depth of 17.94

VERTICAL BEARING CAPACITY: Vertical Loading=0.0, Resistance=55.6, Vertical Factor of Safety=999.00
Request Embedment for Vertical Loading=0.0
Request Total Pile Length=22.0

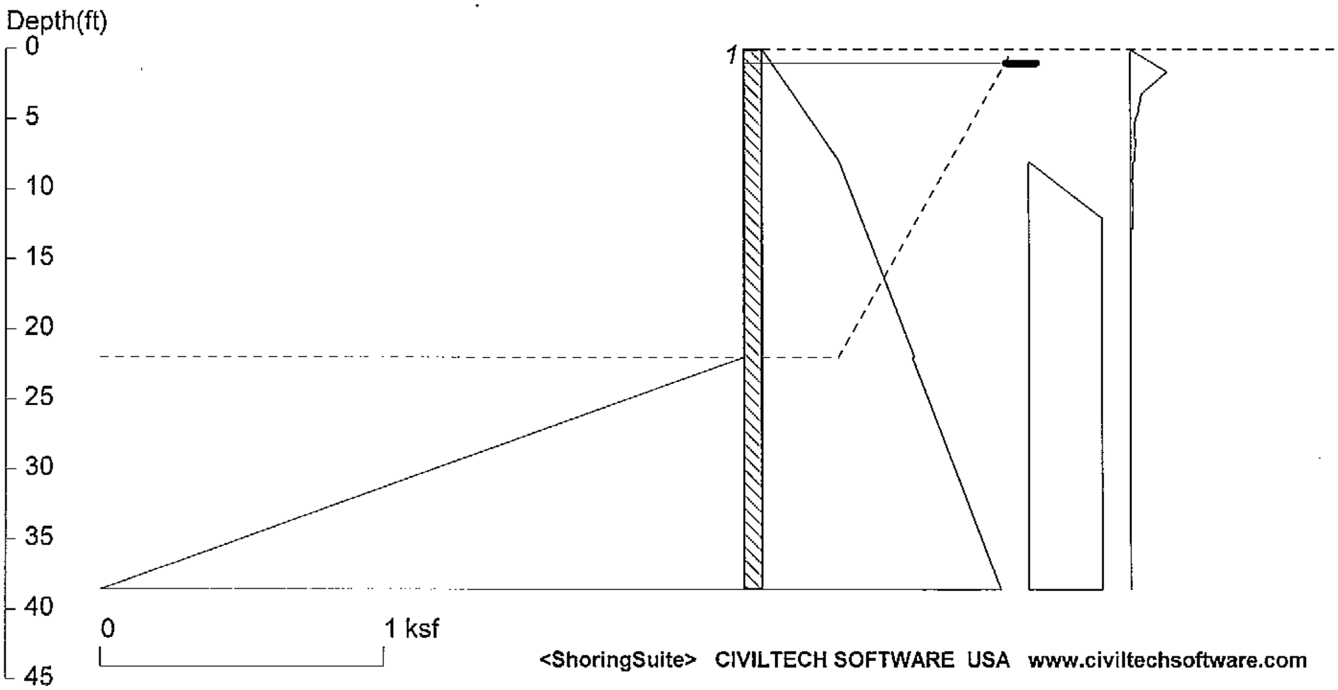
PILE SELECTION:
Request Min. Section Modulus = 25.4 in3/feet, Fy= 36 ksi = 248 MPa, Fb/Fy=1
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
SZ222 (-0.29) SZ24 (-0.27) SZ24A (-0.25) SZ25 (-0.26) CZ114RD (-0.24)
3N(M) (-0.28) PZ27 (-0.26) PLZ23 (-0.23) BZ16.4 (-0.26) RZ10 (-0.28)
134N (-0.26) PZ27 (-0.25) BZ17 (-0.26) SPZ23 (-0.23)

BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Total	Horiz.	Vert.	L_free	Fixed Length
1. Tieback	1.0	0.0	6.8	6.8	0.0	17.6	2.2

Marina Park +10 Seawall

With H 20 Loading



<ShoringSuite> CIVILTECH SOFTWARE USA www.civiltechsoftware.com

Licensed to DBN TerraCosta Consulting Group
Date: 8/6/2008 File Name: C:\Project Files\2500-2599\2573 Marina Park\mp10.sh8

Wall Height=22.0 Pile Diameter=1.0 Pile Spacing=1.0

ACTIVE SPACE:	Z depth	Spacing
1	0.00	1.00
2	22.00	1.00

PASSIVE SPACE:	Z depth	Spacing
1	22.00	1.00

PILE LENGTH: Min. Embedment=16.55, Min. Pile Length=38.55
MOMENT IN PILE: Max. Moment=76.79 at Depth of 17.90

VERTICAL BEARING CAPACITY: Vertical Loading=0.0, Resistance=55.6, Vertical Factor of Safety=999.00
Request Embedment for Vertical Loading=0.0
Request Total Pile Length=22.0

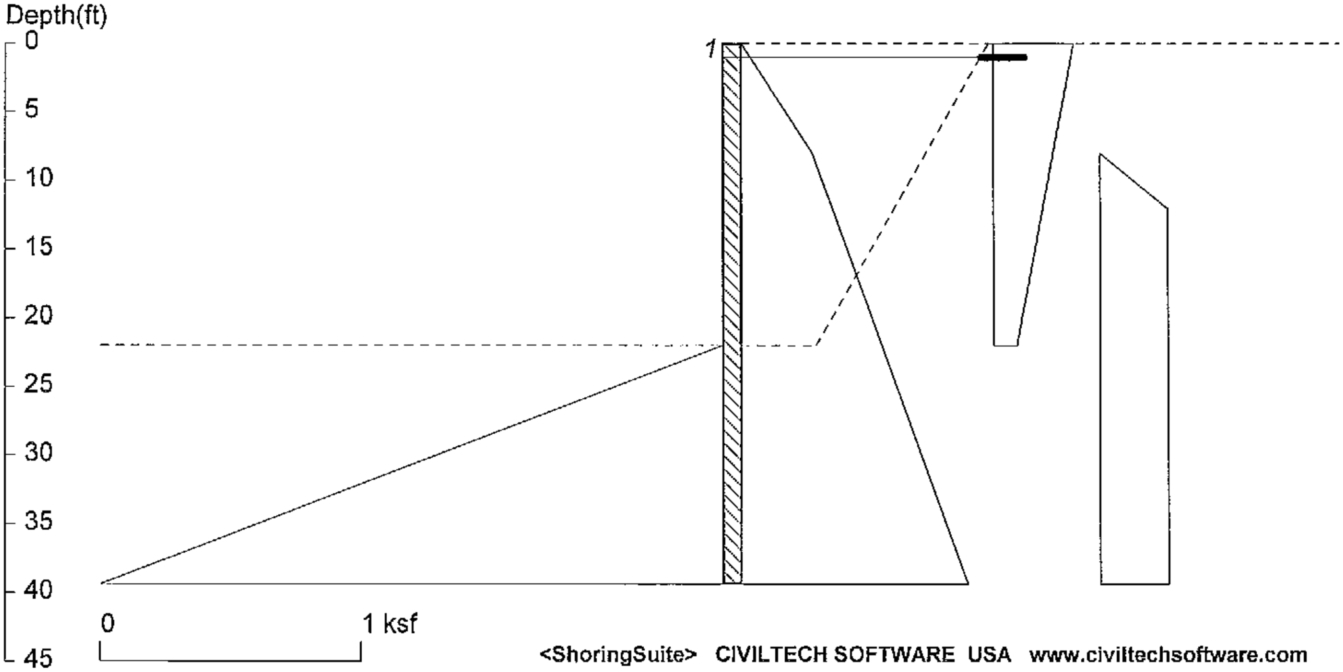
PILE SELECTION:
Request Min. Section Modulus = 25.6 in3/feet, Fy= 36 ksi = 248 MPa, Fb/Fy=1
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
SZ222 (-0.29) SZ24 (-0.27) SZ24A (-0.26) SZ25 (-0.26) CZ114RD (-0.24)
3N(M) (-0.29) PZ27 (-0.26) PLZ23 (-0.24) BZ16.4 (-0.27) RZ10 (-0.28)
134N (-0.27) PZ27 (-0.26) BZ17 (-0.26) SPZ23 (-0.23)

BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Total	Horiz.	Vert.	L_free	Fixed Length
1. Tieback	1.0	0.0	7.1	7.1	0.0	17.6	2.3

Marina Park +10 Seawall

0.30 g



Licensed to DBN TerraCosta Consulting Group

Date: 8/6/2008 File Name: C:\Project Files\2500-2599\2573 Marina Park\3g.sh8

Wall Height=22.0 Pile Diameter=1.0 Pile Spacing=1.0

ACTIVE SPACE:	Z depth	Spacing
1	0.00	1.00
2	22.00	1.00

PASSIVE SPACE:	Z depth	Spacing
1	22.00	1.00

PILE LENGTH: Min. Embedment=17.36, Min. Pile Length=39.36
MOMENT IN PILE: Max. Moment=95.41 at Depth of 17.39

VERTICAL BEARING CAPACITY: Vertical Loading=0.0, Resistance=57.2, Vertical Factor of Safety=999.00
Request Embedment for Vertical Loading=0.0
Request Total Pile Length=22.0

PILE SELECTION:
Request Min. Section Modulus = 48.2 in3/feet, Fy= 36 ksi = 248 MPa, Fb/Fy=0.66
-> Piles meet Min. Section Requirements: Top Deflection is shown in (in)
BZ26 (-0.18) AZ26 (-0.15) H175 (-0.19) PZ35 (-0.17) H215 (-0.15)
BZ32 (-0.15) FSPZ32 (-0.15) PZ40 (-0.12) 5RU3 (-0.16) AZ36 (-0.10)
BZ37 (-0.13) FSPZ38 (-0.12) BZ42 (-0.11) FSPZ45 (-0.10)

BRACE FORCE: Strut, Tieback, Plate Anchor, and Deadman

No. & Type	Depth	Angle	Total	Horiz.	Vert.	L_free	Fixed Length
1. Tieback	1.0	0.0	10.3	10.3	0.0	17.6	3.3

Laterally Loaded Pile Analysis — Marina Park -- 8/05/08

Circular Guide Piles w/L/T=4

Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	1886
Pile Diameter, D (in):	14.00
Pile Modulus, E (psi):	3,000,000
Soil Modulus, f (pci):	15.00
Unsupported Cantilevered Height, H (ft):	22.00
Depth of Embedment, L (ft):	18.00
Effective Depth, T (in):	51.92
Effective Depth, T (ft):	4.33
Lateral Load, P (kips):	2.48
Load Induced Moment, M (Kip-ft):	54.56
Embedment Depth Ratio, L/T:	4.16

Ultimate lateral soil capacity ref: Brom's 1964

Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2

Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4

Soil phi, degrees	32
Soil density, pcf	60
lever arm	22.00
Kp	3.25
Myield,Mtotal(Kip-ft);	77.5

Pult(kips)	3.26	Long Pile
Pult(kips)	16.61	short Pile

Note: Use the smaller of the two

Also note: to obtain the ultimate capacity for a long pile,
you must balance E15 and L13 to obtain the correct answer

Computation of Variation in Soil Induced Moment with L/T = 4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	54.56	0.00	54.56	2430
0.25	1.08	0.992	0.240	54.12	2.58	56.70	2525
0.50	2.16	0.970	0.467	52.92	5.01	57.93	2580
0.75	3.24	0.926	0.627	50.52	6.73	57.25	2550
1.00	4.33	0.859	0.732	46.87	7.85	54.72	2437
1.25	5.41	0.753	0.767	41.08	8.23	49.31	2196
1.50	6.49	0.640	0.747	34.92	8.02	42.93	1912

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE
0.00	0.00	1.56	2.50	0.49	0.15	0.64	"	0.01186740
0.25	1.08	1.16	2.07	0.36	0.13	0.49	"	0.01052344
0.50	2.16	0.82	1.65	0.25	0.10	0.35	"	0.009196804
0.75	3.24	0.52	1.30	0.15	0.08	0.23	"	0.00693808
1.00	4.33	0.30	0.97	0.08	0.06	0.14	"	0.005439049
1.25	5.41	0.12	0.67	0.03	0.04	0.07	"	0.002796966
1.50	6.49	0.03	0.44	0.01	0.03	0.03	"	

Top of Pile Def (in) 6.46 "

NOTE: Top of pile deflection is the combination of:

Ground surface deflection, DEF tot." PLUS	0.64	"
Deflected pile due to angular rotation only, slope*Ht. PLUS	3.13	"
Deflected pile due to loading,PL^3/3EI	2.69	"
where: L=lever arm		

Laterally Loaded Pile Analysis — Marina Park – 8/05/08

Circular Guide Piles w/L/T=4

Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	3217		
Pile Diameter, D (in):	16.00		
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964	
Soil Modulus, f (pci):	15.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2	
Unsupported Cantilevered Height, H (ft):	22.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4	
Depth of Embedment, L (ft):	20.00		
		Soil phi, degrees	32
		Soil density, pcf	60
Effective Depth, **T** (in):	57.77		
Effective Depth, **T** (ft):	4.81		
Lateral Load, P (kips):	3.70	lever arm	22.00
Load Induced Moment, **M** (Kip-ft):	81.40	Kp	3.25
Embedment Depth Ratio, L/T:	4.15	Myield,Mtotal(Kip-ft);	116.6

Pult(kips)	4.85	Long Pile
Pult(kips)	24.79	short Pile

Note: **Use** the smaller of the two

Also note: to obtain the ultimate capacity for a long pile, you must balance E15 and L13 to obtain the correct answer

Computation of Variation in Soil Induced Moment with L/T = 4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	81.40	0.00	81.40	2429
0.25	1.20	0.992	0.240	80.75	4.27	85.02	2537
0.50	2.41	0.970	0.467	78.96	8.32	87.28	2604
0.75	3.61	0.926	0.627	75.38	11.17	86.54	2583
1.00	4.81	0.859	0.732	69.92	13.04	82.96	2476
1.25	6.02	0.753	0.767	61.29	13.66	74.96	2237
1.50	7.22	0.640	0.747	52.10	13.31	65.40	1952

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE
0.00	0.00	1.56	2.50	0.53	0.18	0.71	"	0.01177281
0.25	1.20	1.16	2.07	0.39	0.15	0.54	"	0.01045962
0.50	2.41	0.82	1.65	0.27	0.12	0.39	"	0.009132189
0.75	3.61	0.52	1.30	0.16	0.10	0.26	"	0.006923528
1.00	4.81	0.30	0.97	0.09	0.07	0.16	"	0.005449051
1.25	6.02	0.12	0.67	0.03	0.05	0.08	"	0.002841366
1.50	7.22	0.03	0.44	0.01	0.03	0.04	"	

Top of Pile Def (in) 6.17 "

NOTE: Top of pile deflection is the combination of:

Ground surface deflection, **DEF** tot." PLUS	0.71	"
Deflected pile due to angular rotation only, slope*Ht. PLUS	3.11	"
Deflected pile due to loading,PL^3/3EI	2.35	"

where: L=lever arm

Laterally Loaded Pile Analysis --- Marina Park – 8/05/08

Circular Guide Piles w/L/T=4

Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	7854
Pile Diameter, D (in):	20.00
Pile Modulus, E (psi):	3,000,000
Soil Modulus, f (pci):	15.00
Unsupported Cantilevered Height, H (ft):	22.00
Depth of Embedment, L (ft):	23.00
Effective Depth, T (in):	69.06
Effective Depth, T (ft):	5.75
Lateral Load, P (kips):	7.25
Load Induced Moment, M (Kip-ft):	159.50
Embedment Depth Ratio, L/T:	4.00

Ultimate lateral soil capacity ref: Brom's 1964

Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2

Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4

Soil phi, degrees	32
Soil density, pcf	60
lever arm	22.00
Kp	3.25
Myield,Mtotal(Kip-ft);	232.5

Pult(kips)	9.47	Long Pile
Pult(kips)	43.99	short Pile

Note: Use the smaller of the two

Also note: to obtain the ultimate capacity for a long pile, you must balance E15 and L13 to obtain the correct answer

Computation of Variation in Soil Induced Moment with L/T = 4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	159.50	0.00	159.50	2437
0.25	1.44	0.992	0.240	158.22	10.01	168.24	2570
0.50	2.88	0.970	0.467	154.72	19.48	174.20	2662
0.75	4.32	0.926	0.627	147.70	26.16	173.86	2656
1.00	5.75	0.859	0.732	137.01	30.54	167.55	2560
1.25	7.19	0.753	0.767	120.10	32.00	152.11	2324
1.50	8.63	0.640	0.747	102.08	31.17	133.25	2036

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"		SLOPE
0.00	0.00	1.56	2.50	0.60	0.25	0.86	"	0.01170813
0.25	1.44	1.16	2.07	0.45	0.21	0.66	"	0.01043860
0.50	2.88	0.82	1.65	0.31	0.17	0.48	"	0.009097804
0.75	4.32	0.52	1.30	0.19	0.13	0.32	"	0.0069595
1.00	5.75	0.30	0.97	0.10	0.10	0.20	"	0.005516017
1.25	7.19	0.12	0.67	0.04	0.07	0.10	"	0.002946886
1.50	8.63	0.03	0.44	0.01	0.04	0.05	"	

Top of Pile Def (in) 5.84 "

NOTE: Top of pile deflection is the combination of:

Ground surface deflection, DEF tot." PLUS	0.86	"
Deflected pile due to angular rotation only, slope*Ht. PLUS	3.09	"
Deflected pile due to loading,PL^3/3EI	1.89	"

where: L=lever arm

Laterally Loaded Pile Analysis --- Marina Park -- 8/05/08

Circular Guide Piles w/L/T=4

Reese & Matlock solution - DM7.02

Pile Moment of Inertia, I (in^4):	16286		
Pile Diameter, D (in):	24.00		
Pile Modulus, E (psi):	3,000,000	Ultimate lateral soil capacity ref: Brom's 1964	
Soil Modulus, f (pci):	15.00	Pult=0.5*soil-density*D*L^3*Kp/(H+L) for L/T<2	
Unsupported Cantilevered Height, H (ft):	22.00	Pult=M/(H+0.54(P/soil-density*D*Kp)^0.5) for L/T>4	
Depth of Embedment, L (ft):	27.00		
		Soil phi, degrees	32
		Soil density, pcf	60
Effective Depth, T (in):	79.90		
Effective Depth, T (ft):	6.66		
Lateral Load, P (kips):	13.70	lever arm	22.00
Load Induced Moment, M (Kip-ft):	301.40	Kp	3.25
Embedment Depth Ratio, L/T:	4.05	Myield,Mtotal(Kip-ft);	447.3

Pult(kips)	17.75	Long Pile
Pult(kips)	78.43	short Pile

Note: Use the smaller of the two

Also note: to obtain the ultimate capacity for a long pile, you must balance E15 and L13 to obtain the correct answer

Computation of Variation in Soil Induced Moment with L/T = 4

Depth,T	Depth,ft	Fmm	Fpt	Mm	Mpt	Mtotal	Fiber Bending, Fb (psi)
0.00	0.00	1.000	0.000	301.40	0.00	301.40	2665
0.25	1.66	0.992	0.240	298.99	21.89	320.88	2837
0.50	3.33	0.970	0.467	292.36	42.60	334.96	2962
0.75	4.99	0.926	0.627	279.10	57.20	336.29	2973
1.00	6.66	0.859	0.732	258.90	66.78	325.68	2880
1.25	8.32	0.753	0.767	226.95	69.97	296.92	2625
1.50	9.99	0.640	0.747	192.90	68.14	261.04	2308

Computation of Pile Deformation with L/T = 4

Depth, T	Depth, ft	Fdm	Fdp	DEF.m	DEF.pt	DEF tot,"	SLOPE
0.00	0.00	1.56	2.50	0.74	0.36	1.09 "	0.01276292
0.25	1.66	1.16	2.07	0.54	0.30	0.84 "	0.01141461
0.50	3.33	0.82	1.65	0.38	0.24	0.61 "	0.009932853
0.75	4.99	0.52	1.30	0.23	0.19	0.41 "	0.007658796
1.00	6.66	0.30	0.97	0.12	0.14	0.26 "	0.006107636
1.25	8.32	0.12	0.67	0.04	0.10	0.14 "	0.003330721
1.50	9.99	0.03	0.44	0.01	0.06	0.07 "	

Top of Pile Def (in) 6.18 "

NOTE: Top of pile deflection is the combination of:

Ground surface deflection, DEF tot." PLUS	1.09 "
Deflected pile due to angular rotation only, slope*Ht. PLUS	3.37 "
Deflected pile due to loading,PL^3/3EI	1.72 "
where: L=lever arm	

Naval Facilities Engineering Command
200 Stovall Street
Alexandria, Virginia 22332-2300

Foundations & Earth Structures

DESIGN MANUAL 7.02
REVALIDATED BY CHANGE 1 SEPTEMBER 1986

Section 7. LATERAL LOAD CAPACITY

1. DESIGN CONCEPTS. A pile loaded by lateral thrust and/or moment at its top, resists the load by deflecting to mobilize the reaction of the surrounding soil. The magnitude and distribution of the resisting pressures are a function of the relative stiffness of pile and soil.

Design criteria is based on maximum combined stress in the piling, allowable deflection at the top or permissible bearing on the surrounding soil. Although 1/4-inch at the pile top is often used as a limit, the allowable lateral deflection should be based on the specific requirements of the structure.

2. DEFORMATION ANALYSIS - SINGLE PILE.

a. General. Methods are available (e.g., Reference 9 and Reference 31, Non-Dimensional Solutions for Laterally Loaded Piles, with Soil Modulus Assumed Proportional to Depth, by Reese and Matlock) for computing lateral pile load-deformation based on complex soil conditions and/or non-linear soil stress-strain relationships. The COM 622 computer program (Reference 32, Laterally Loaded Piles: Program Documentation, by Reese) has been documented and is widely used. Use of these methods should only be considered when the soil stress-strain properties are well understood.

Pile deformation and stress can be approximated through application of several simplified procedures based on idealized assumptions. The two basic approaches presented below depend on utilizing the concept of coefficient of lateral subgrade reaction. It is assumed that the lateral load does not exceed about 1/3 of the ultimate lateral load capacity.

b. Granular Soil and Normally to Slightly Overconsolidated Cohesive Soils. Pile deformation can be estimated assuming that the coefficient of subgrade reaction, K_h, increases linearly with depth in accordance with:

$$K_h = \frac{fz}{D}$$

where: K_h = coefficient of lateral subgrade reaction (tons/ft^3)

f = coefficient of variation of lateral subgrade reaction (tons/ft^3)

z = depth (feet)

D = width/diameter of loaded area (feet)

Guidance for selection of f is given in Figure 9 for fine-grained and coarse-grained soils.

c. Heavily Overconsolidated Cohesive Soils. For heavily overconsolidated hard cohesive soils, the coefficient of lateral subgrade reaction can be assumed to be constant with depth. The methods presented in Chapter 4 can be used for the analysis; K_h varies between 35c and 70c (units of force/length^3) where c is the undrained shear strength.

d. Loading Conditions. Three principal loading conditions are illustrated with the design procedures in Figure 10, using the influence diagrams of Figure 11, 12 and 13 (all from Reference 31). Loading may be limited by allowable deflection of pile top or by pile stresses.

Case I. Pile with flexible cap or hinged end condition. Thrust and moment are applied at the top, which is free to rotate. Obtain total deflection, moment, and shear in the pile by algebraic sum of the effects of thrust and moment, given in Figure 11.

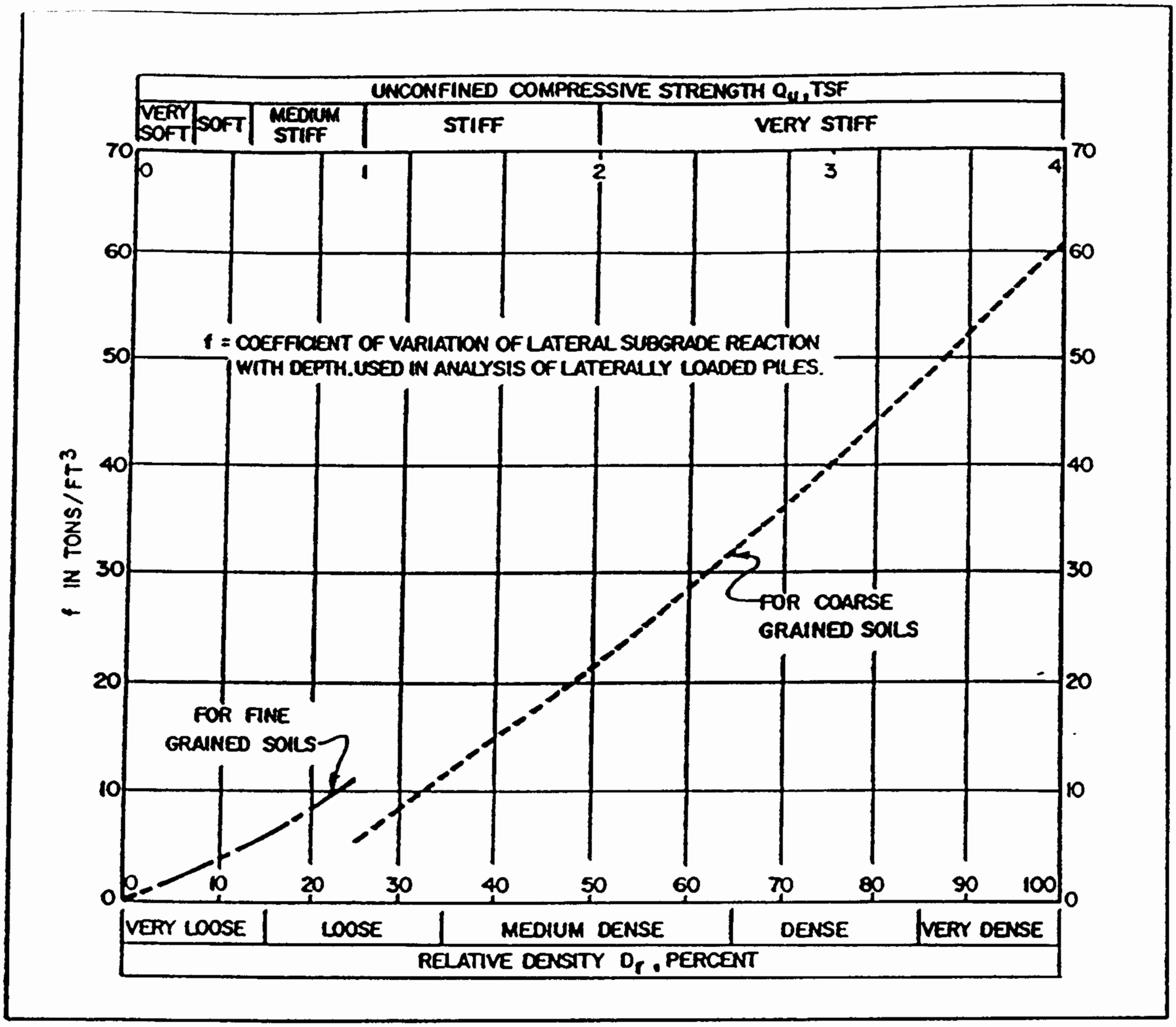


FIGURE 9
Coefficient of Variation of Subgrade Reaction

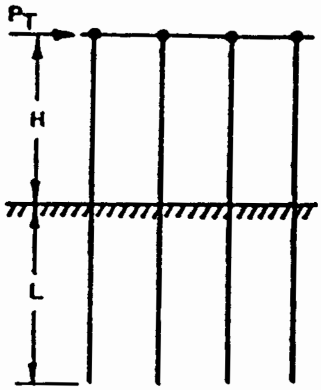
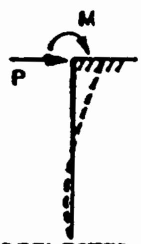
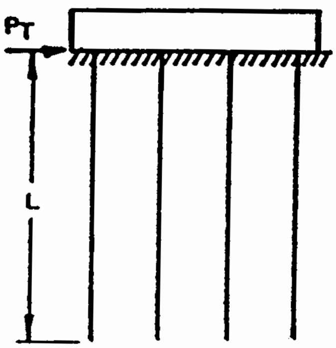
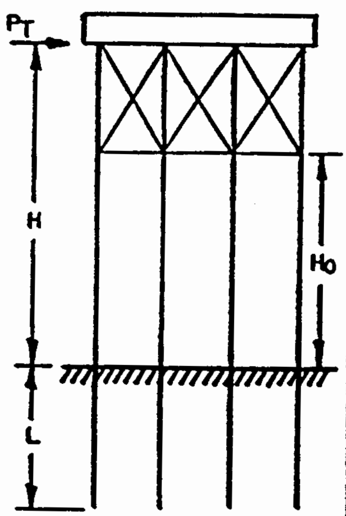
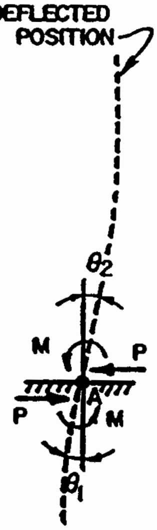
CONDITION	LOAD AT GROUND LINE	DESIGN PROCEDURE
CASE I. FLEXIBLE CAP, ELEVATED POSITION		
 n = NUMBER OF PILES	FOR EACH PILE: $P = \frac{P_T}{n}$ $M = PH$  DEFLECTED POSITION	FOR DEFINITION OF PARAMETERS SEE FIGURE 12 1. COMPUTE RELATIVE STIFFNESS FACTOR. $T = (\frac{EI}{f})^{1/5}$ 2. SELECT CURVE FOR PROPER $\frac{L}{T}$ IN FIGURE 11. 3. OBTAIN COEFFICIENTS F_δ, F_M, F_V AT DEPTHS DESIRED. 4. COMPUTE DEFLECTION, MOMENT AND SHEAR AT DESIRED DEPTHS USING FORMULAS OF FIGURE 11. NOTE: "f" VALUES FROM FIGURE 9 AND CONVERT TO LB/IN^3.
CASE II. PILES WITH RIGID CAP AT GROUND SURFACE		
		1. PROCEED AS IN STEP 1, CASE I. 2. COMPUTE DEFLECTION AND MOMENT AT DESIRED DEPTHS USING COEFFICIENTS F_δ, F_M AND FORMULAS OF FIGURE 12. 3. MAXIMUM SHEAR OCCURS AT TOP OF PILE AND EQUALS $P = \frac{P_T}{n}$ IN EACH PILE.
CASE III. RIGID CAP, ELEVATED POSITION		
	DEFLECTED POSITION 	1. ASSUME A HINGE AT POINT A WITH A BALANCING MOMENT M APPLIED AT POINT A. 2. COMPUTE SLOPE θ_2 ABOVE GROUND AS A FUNCTION OF M FROM CHARACTERISTICS OF SUPERSTRUCTURE. 3. COMPUTE SLOPE θ_1 FROM SLOPE COEFFICIENTS OF FIGURE 13 AS FOLLOWS: $\theta_1 = F_\theta (\frac{PT^2}{EI}) + F_\theta (\frac{MT}{EI})$ 4. EQUATE $\theta_1 = \theta_2$ AND SOLVE FOR VALUE OF M. 5. KNOWING VALUES OF P AND M, SOLVE FOR DEFLECTION, SHEAR, AND MOMENT AS IN CASE I. NOTE: IF GROUND SURFACE AT PILE LOCATION IS INCLINED, LOAD P TAKEN BY EACH PILE IS PROPORTIONAL TO I/H_0^3.

FIGURE 10
Design Procedure for Laterally Loaded Piles

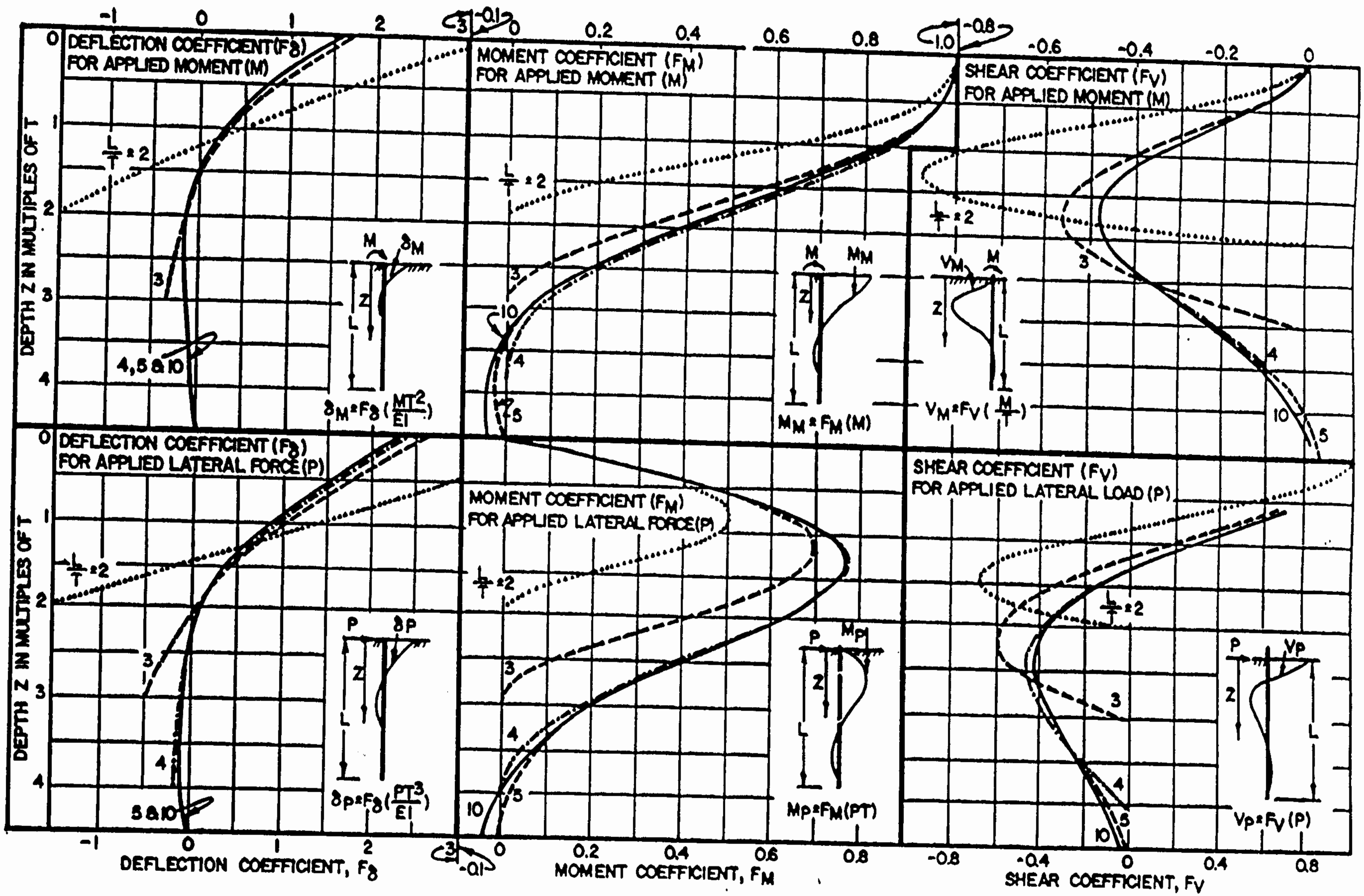


FIGURE 11
Influence Values for Pile with Applied Lateral Load and Moment
(Case I. Flexible Cap or Hinged End Condition)

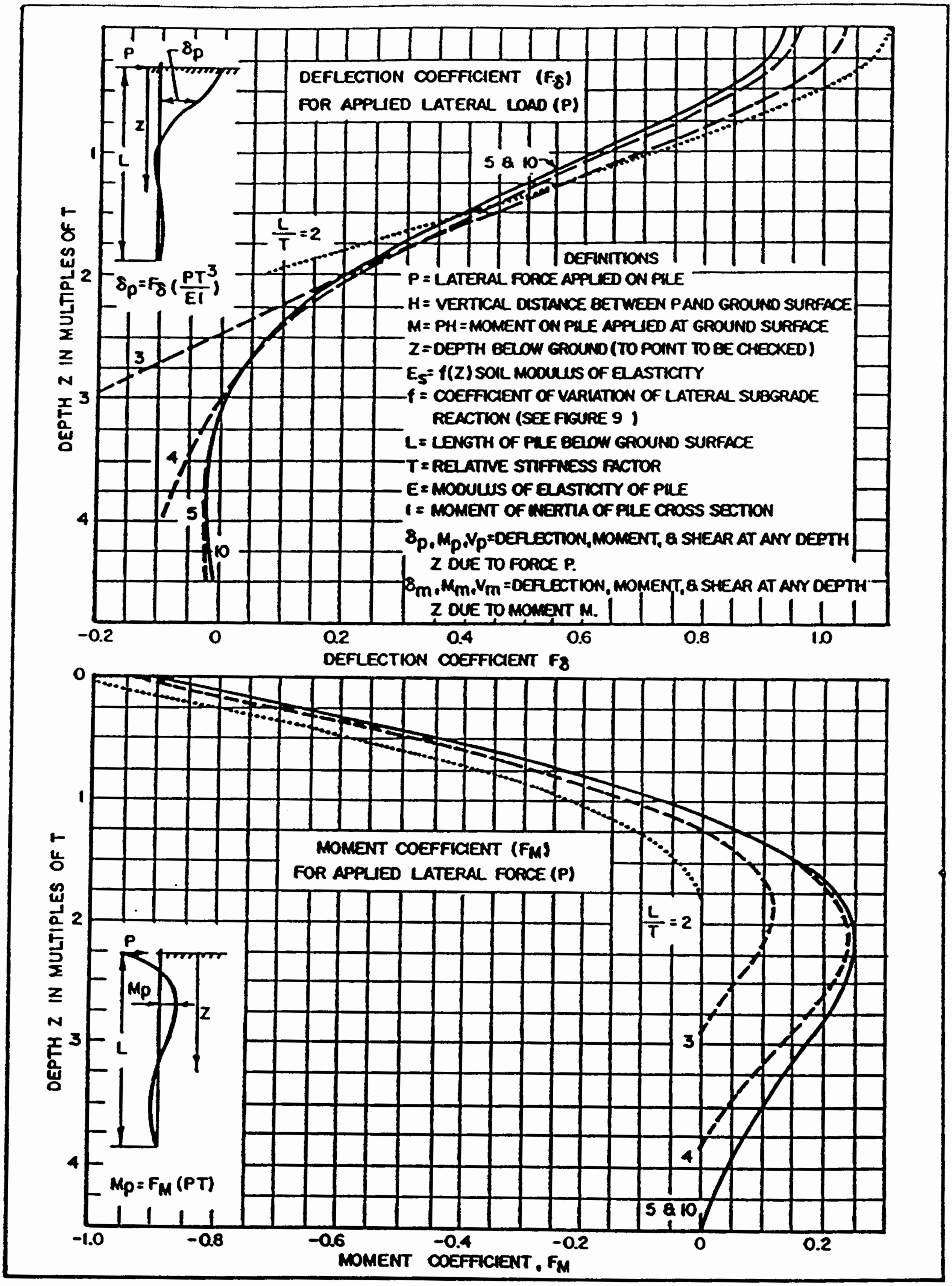


FIGURE 12
Influence Values for Laterally Loaded Pile
(Case II. Fixed Against Rotation at Ground Surface)

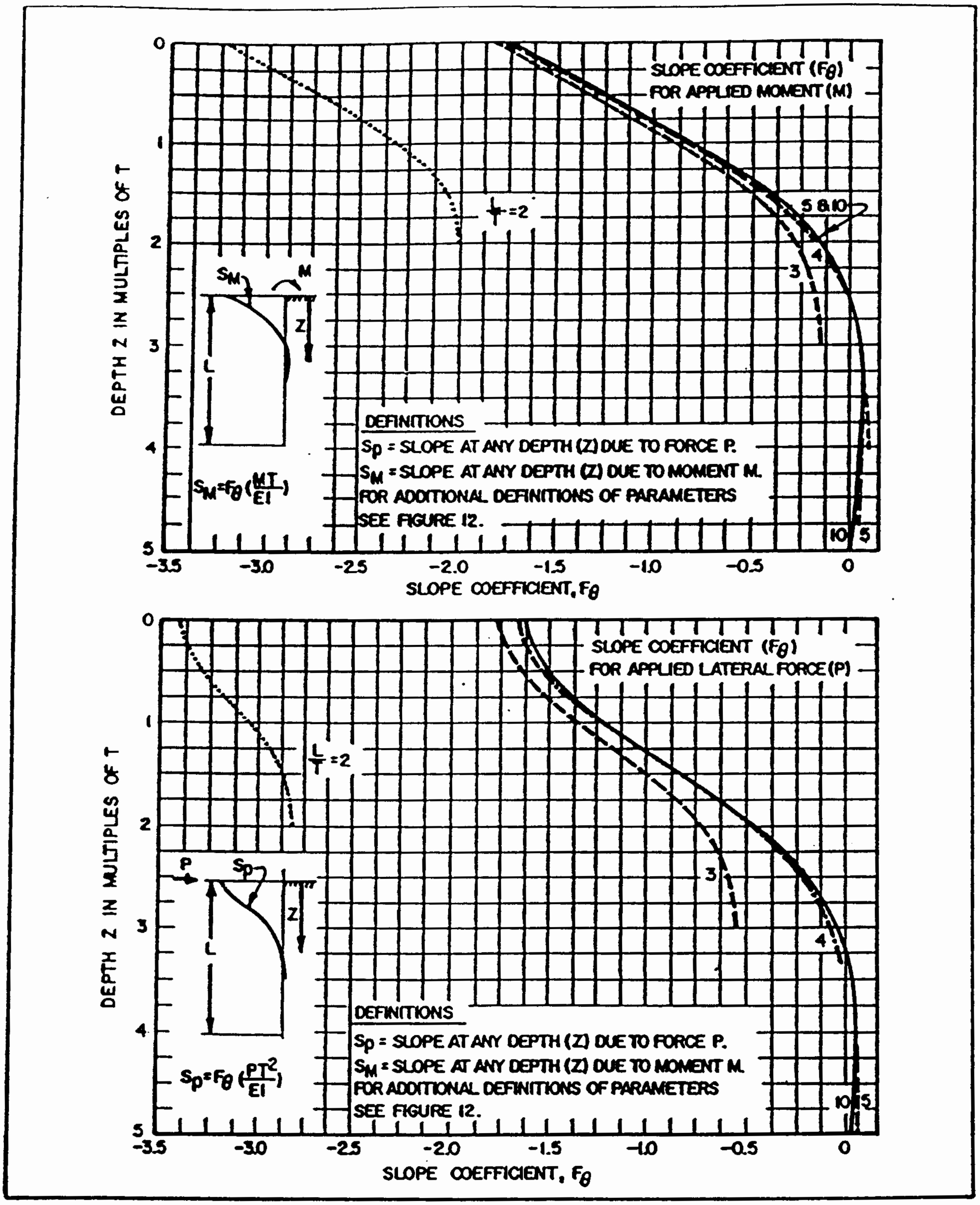


FIGURE 13
Slope Coefficient for Pile with Lateral Load or Moment

Case II. Pile with rigid cap fixed against rotation at ground surface. Thrust is applied at the top, which must maintain a vertical tangent. Obtain deflection and moment from influence values of Figure 12.

Case III. Pile with rigid cap above ground surface. Rotation of pile top depends on combined effect of superstructure and resistance below ground. Express rotation as a function of the influence values of Figure 13 and determine moment at pile top. Knowing thrust and moment applied at pile top, obtain total deflection, moment and shear in the pile by algebraic sum of the separate effects from Figure 11.

3. CYCLIC LOADS.

Lateral subgrade coefficient values decrease to about 25% the initial value due to cyclic loading for soft/loose soils and to about 50% the initial value for stiff/dense soils.

4. LONG-TERM LOADING. Long-term loading will increase pile deflection corresponding to a decrease in lateral subgrade reaction. To approximate this condition reduce the subgrade reaction values to 25% to 50% of their initial value for stiff clays, to 20% to 30% for soft clays, and to 80% to 90% for sands.

5. ULTIMATE LOAD CAPACITY - SINGLE PILES. A laterally loaded pile can fail by exceeding the strength of the surrounding soil or by exceeding the bending moment capacity of the pile resulting in a structural failure. Several methods are available for estimating the ultimate load capacity.

The method presented in Reference 33, Lateral Resistance of Piles in Cohesive Soils, by Broms, provides a simple procedure for estimating ultimate lateral capacity of piles.

6. GROUP ACTION. Group action should be considered when the pile spacing in the direction of loading is less than 6 to 8 pile diameters. Group action can be evaluated by reducing the effective coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R (Reference 9) as follows:

Pile Spacing in Direction of Loading D = Pile Diameter	Subgrade Reaction Reduction Factor R
8D	1.00
6D	0.70
4D	0.40
3D	0.25

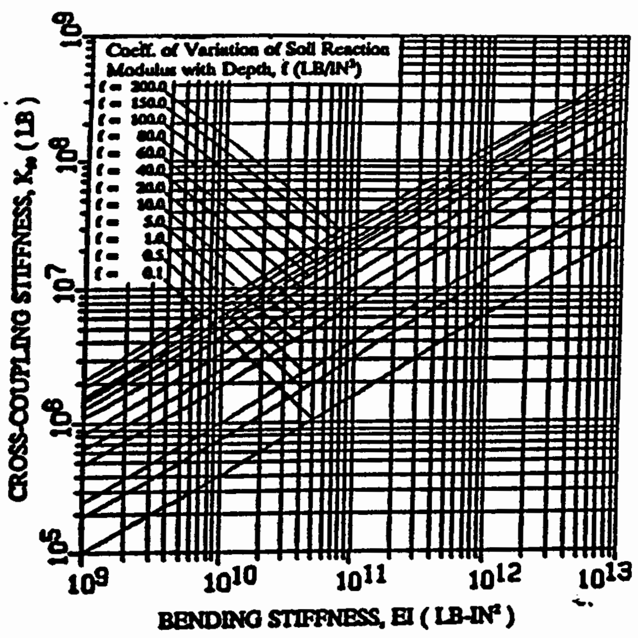


Figure 9. Pile Cross-Coupling Stiffness, $K_{\delta\theta}$

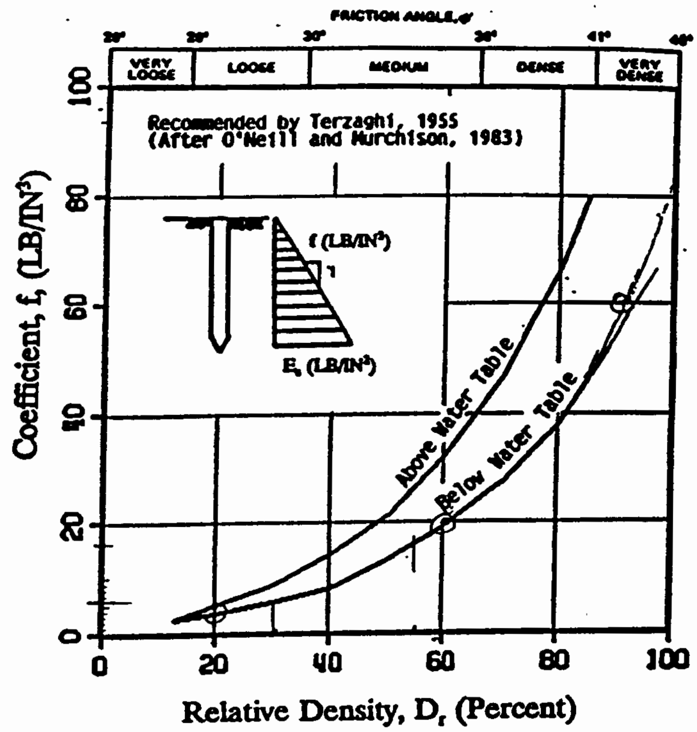


Figure 10. Recommendations for Coefficient f for Sands (Note: 1 LB/IN^3 = 0.27 N/cm^3)

he authors. This recommendation and results of the correlation for clay are shown in Figure 11. Only the upper five iameters of soils (soil type and ground ater) need to be considered in usage of the presented design charts.

<u>Limitations of Approach</u>. There are everal simplifying assumptions in the presented approach. The coefficient f is not an intrinsic soil parameter. The ecommendations for f presented in Figures) and 11 are appropriate for piles in typical highway bridge foundations (i.e. smaller piles). Furthermore, the embedment ffect has not been taken into account in he procedure. Therefore the recommenda-tions are conservative and appropriate for shallow embedment conditions (say less than feet or 1.5 m).

Although correlations for the coefficient f can be conducted for other conditions e.g. larger piles and bigger embedment epths), the additional complexity negates he merits of the use of simplified linear elastic solutions. For such cases, com-uter solutions, which can readily accomo-ate nonlinear effects and more general oundary conditions, are recommended.

<u>Comparison to Caltrans Practice</u>. The ove procedure can be compared to the ractice adopted by Caltrans. In Caltrans

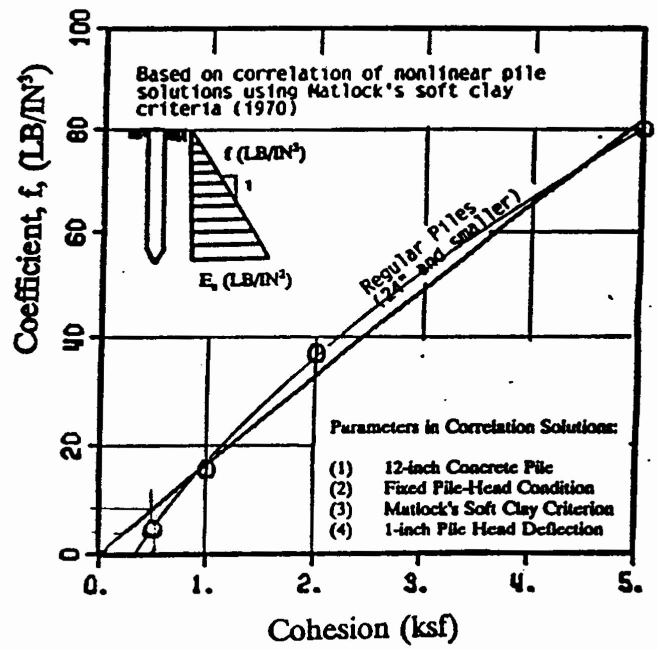


Figure 11. Recommendations of Coefficient f for Clays (Note: 1 LB/IN^3 = 0.27 N/cm^3)

APPENDIX E

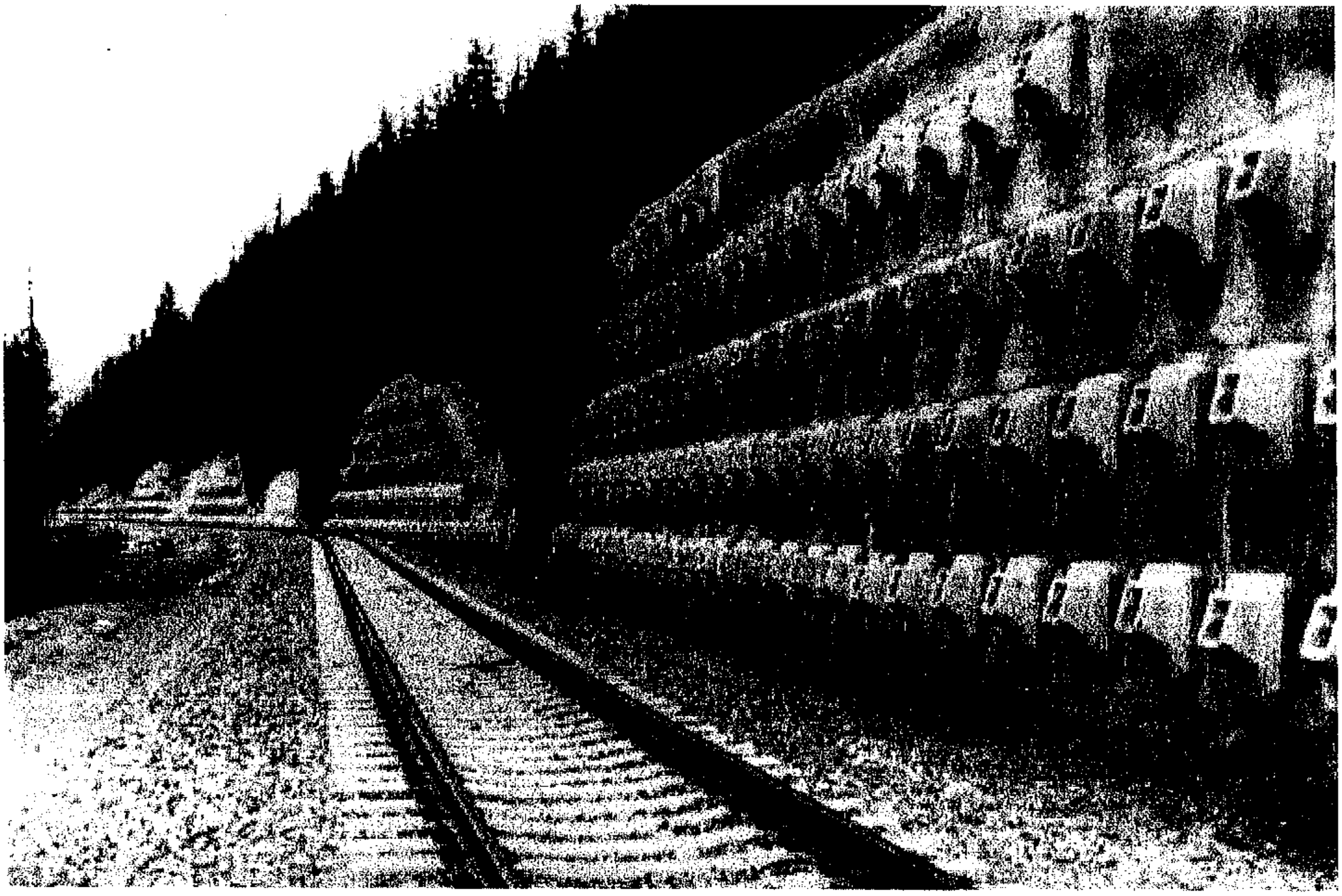
DSI PRODUCT LITERATURE

TerraCosta
Consulting Group

DYWIDAG-SYSTEMS INTERNATIONAL

DSI

DYWIDAG Rock and Soil Anchors


Rogers Pass Slope Stabilization, British Columbia, Canada

Unternehmensgruppe DYWIDAG

DYWIDAG Threadbar Rock and Soil Anchors

Dywidag Systems International was a pioneer in the development of rock and soil anchor systems and technology. Today DSI is a world leader in this field with an outstanding reputation of product quality and customer service. The double corrosion protected THREADBAR® anchor is universally recognized as the "standard" for anchor performance and corrosion protection. DSI is dedicated to the advancement of the "State-of-the-Art" for rock and soil anchors and stands ready to support you during the design, planning and construction of your project. When questions arise, contact your nearest DSI representative.

One Source for Bar and Strand Anchors

DSI offers a complete line of THREADBARS® and multistrand anchors designed for both temporary or permanent use, manufactured from materials best suited to meet the needs of your project.

THREADBAR® Anchors are available in 1" (26 mm), 1-1/4" (32 mm), and 1-3/8" (36 mm) nominal diameter, in lengths up to 60 feet (18.3 m) without couplers, with a guaranteed minimum ultimate tensile stress of 150 or 160 ksi (1034 or 1103 MPa).

Multistrand Anchors manufactured from 0.6" dia. (15.2 mm) 270k (1861 MPa) strand are available in sizes up to 61 strands. Larger anchors are available but system components are not stocked.

Rock Bolts and *Soil Nails* manufactured from ASTM A615 grade 60 are produced in sizes ranging from #6 up to, and including, #11 grade 75 bars. Special steels for high impact, seismic and low temperature applications can be made available on special order.

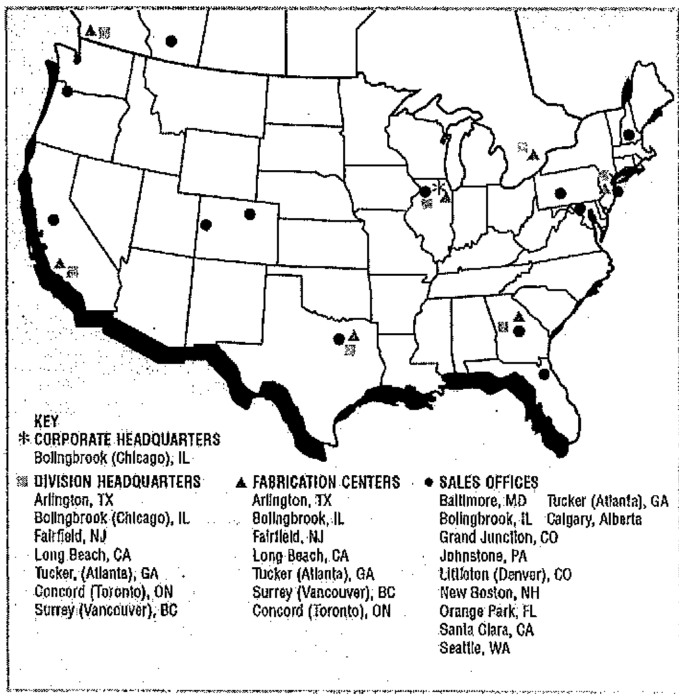
The DSI Advantage

As a full service organization, DSI is prepared to supply design assistance and practical field know-how. This service can also be used to optimize the design process by helping to select the anchor system best suited to meet specific project requirements.

The regional warehouse and fabricating centers strategically located throughout North America, coupled with an extensive network of local sales/service centers, provide prompt, reliable response to customer needs. Most orders can be supplied from inventory with short lead time.

To minimize site labor and to optimize quality control, a variety of shop prefabricating services are available for both bar and strand anchors. In many cases the anchors can be delivered to the site ready for immediate installation without the need for site assembly. The application of corrosion protection grouting at the job site can also be minimized and, in many cases, completely eliminated, saving time and money.

In some locations both supply and installation, including drilling services, are available for any size project.



Whatever your needs you can count on DSI for *quality* from start to finish. The dedication of our staff to quality and service will help you complete your project successfully and on time.

Applications

Prestressed rock and soil anchors have become an important tool for the geotechnical engineer. Their safe and reliable use in both permanent and temporary applications is accepted throughout the world.

Soil Anchors are pressure grouted anchors installed in either cohesive or non-cohesive soil or loose rock. The anchors transfer forces into the ground by means of a steel tendon and a well defined grout body. In the free stressing length the anchor remains free to move.

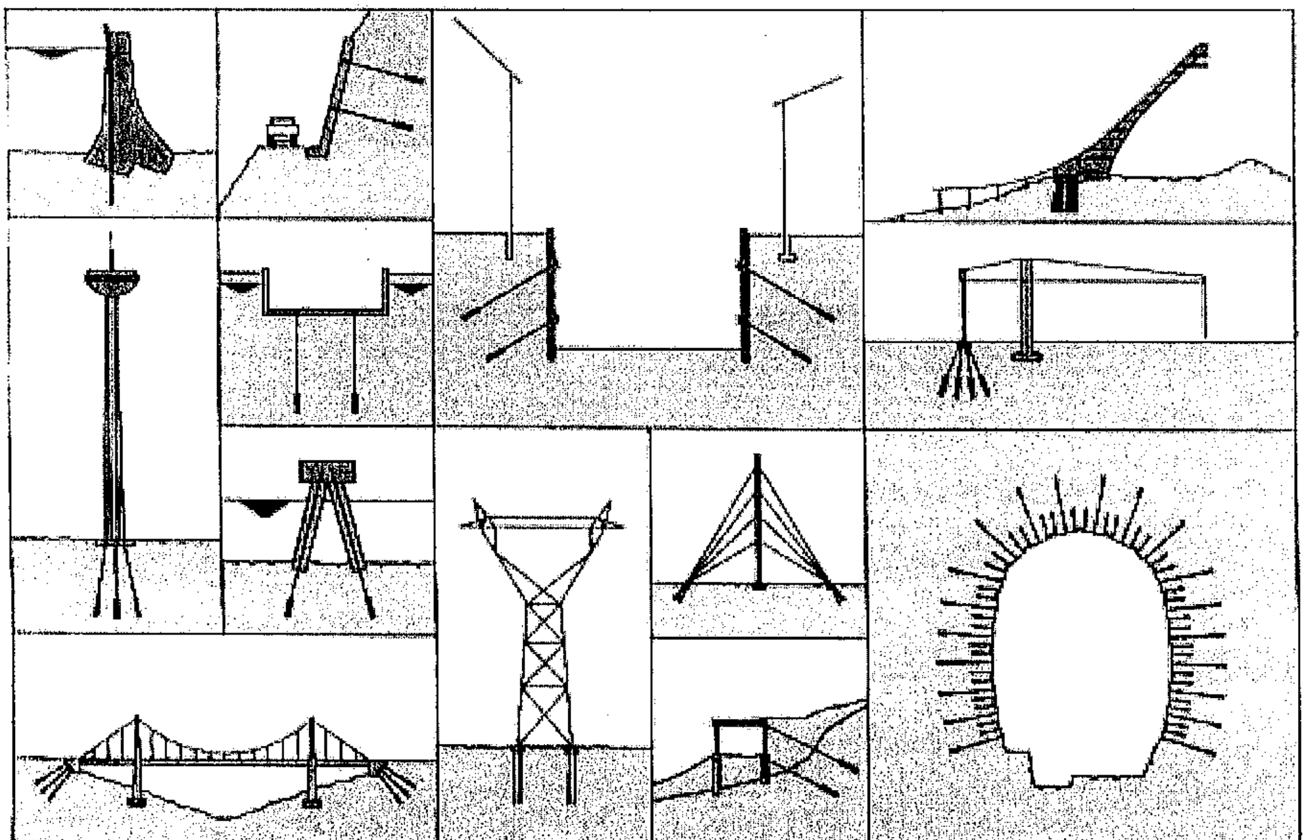
Soil anchors are generally used to:

- anchor support structures for excavations such as sheet pile walls, soldier piles and lagging, drilled piles and slurry walls.
- counteract uplift forces in structures subjected to buoyancy and lateral loads.
- transfer external forces to the ground; e.g.,wind, earthquake .
- stabilize eccentrically loaded foundations.
- stabilize material or excavated slopes.

Rock Anchors are post-tensioned tendons installed in drilled holes for which at least the entire bond length is located in rock. The anchor force is transmitted to the rock by bond between the grout body and the rock. Rock anchors can remain unbonded in the free stressing length allowing the anchor to be checked and retensioned at any time. In such cases, adequate corrosion protection for the stressing anchorage and the free stressing length must be provided. On the other hand, the free stressing length can also be fully grouted after the anchor has been stressed, in which case force adjustment is no longer possible.

Rock anchors are generally used to:

- anchor external forces and uplift forces.
- anchor retaining walls.
- stabilize eccentrically loaded foundations, slopes, rock walls and cuts.
- stabilize underground excavations and mines.
- increase the stability of dams.



Threadbar® Anchors

The Dywidag Threadbar Rock and Soil Anchor System is manufactured in the United States and Canada exclusively by Dywidag Systems International.

Simple and Rugged

The threadbar has a continuous rolled-on pattern of deformations along its entire length which allows anchorage hardware or couplers to thread onto the bar at any point. The coarse thread is almost indestructible under normal job site conditions.

Positive Anchorage

The bar is anchored using a threaded nut which, unlike a wedge type anchorage, is not liable to be loosened when the anchor force is reduced due to possible ground movements. In addition, the threaded nut anchorage has a known overload capacity which cannot be duplicated by a wedge type anchorage without the utilization of elaborate and expensive details.

Easy to Stress

The reliable and compact threaded nut anchorage has almost no anchor set. Its hemispherical shape easily accommodates the small angular misalignments between bar and anchorage due to construction tolerances. Lightweight, durable equipment makes stressing, restressing and adjusting the anchor load up or down easy to do.

Easy to Check Actual Prestress Load and Restress

The threaded design makes it possible to make a lift off test and/or adjust the anchor load at any time during the service life of the anchor. Corrosion protection can be maintained at all times.

Easy to Splice

The continuous thread makes it possible to extend the threadbar to any length, simplifying transportation and installation. Extending the bar beyond the stressing end to connect to another structural member is also a simple operation.

DSI reserves the right to change the design or details of its products without notice. Specific information for job details and drawings should be obtained from your DSI Sales Engineer.

High Bond Strength

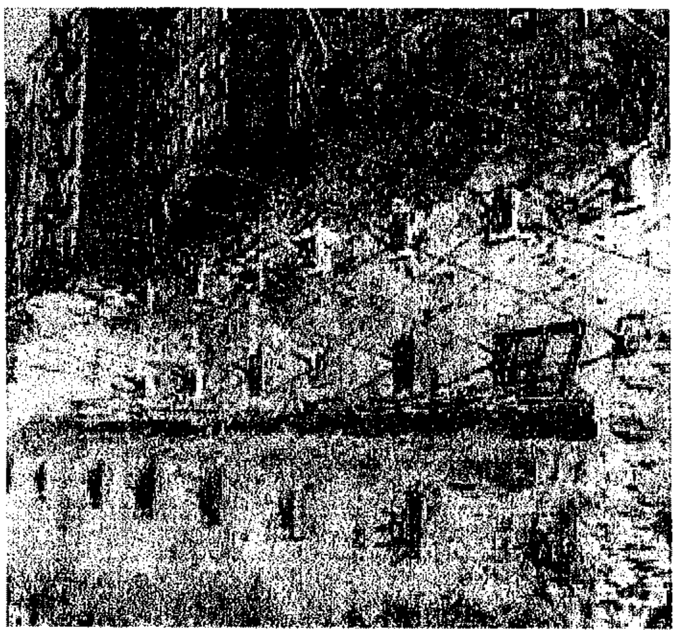
The deformation pattern provides excellent bond between the bar and cement grout making it possible to reliably transfer anchor prestress load into the grout without the need for additional mechanical devices. The narrow spacing of the deformations assures close crack spacing in the surrounding grout and therefore smaller crack widths which will not degrade the corrosion protection.

Removable

The threadbar can be removed after destressing the anchor by unscrewing the unbonded portion of the bar from a coupler or out of an embedded end anchorage. Bars with end anchors and sleeved within the bond length can be completely removed. This is especially important where temporary anchors are installed below adjacent properties and must be removed after use.

Easy to Install

Because of their inherent stiffness and ruggedness, threadbar anchors can be easily installed in any position, including upward. It is particularly easy to install a bar anchor in a pre-grouted hole.



Public school No. 48, New York City Board of Education, Manhattan. Permanent DCP anchors extended to support subsequent retaining wall.

Insurance Against Anchor Failure

In cohesive and other poor soils, a proven and reliable DSI post-grouting system can be used to increase the capacity of an anchor. The use of this system can make the difference between an anchor that works and one that does not.

Corrosion Protection Options

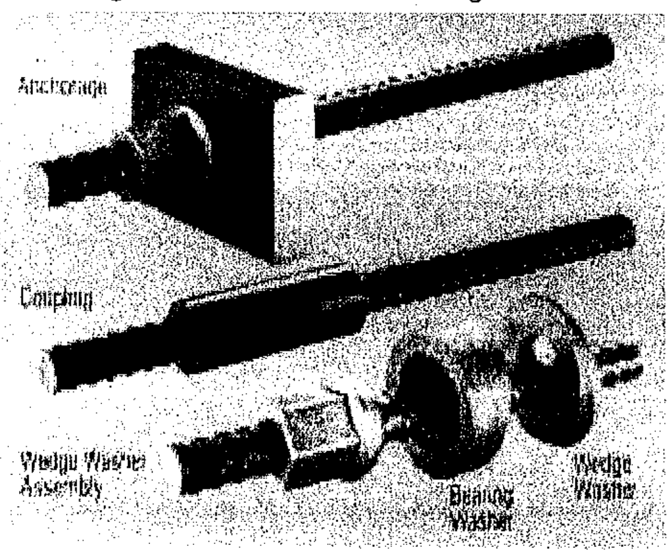
A wide variety of corrosion protection options are available to choose from depending upon the expected length of service and the aggressiveness of the environment.

Unprotected Anchors

Unprotected anchors are used for temporary applications. The free stressing length is unprotected while the bond length is embedded in the cement grout body. Unprotected anchors may be subject to corrosion. However, the relatively large diameter and solid cross section of the Dywidag threadbar offers more corrosion resistance than smaller diameter high strength, prestressing steel elements with a larger surface area.

Single Corrosion Protected Anchors SCP

Single corrosion protected anchors are used for temporary anchors and sometimes for permanent anchors in non-aggressive rock or soil. A polyethylene sheathing covers the free stressing length. The threadbar is coated with a corrosion inhibitor before the polyethylene sheathing is installed. The bond length is covered with cement grout.



Double Corrosion Protected Anchors DCP

Double corrosion protected anchors are recommended for anchors with a long service life and for an environment where aggressive materials or stray electrical currents are expected.

A corrugated high strength PVC sheathing with plastic end caps is installed over the full length of the anchor. The annular space between threadbar and PVC is fully grouted before the anchor is installed. To accommodate the bar elongation during stressing, a short length of threadbar is left free of the corrugated sheathing under the stressing anchorage. A steel pipe welded to the anchor plate and filled with corrosion preventive compound or grout protects the free end of the bar against corrosion.

A smooth plastic sheathing is installed over the corrugated sheathing in the stressing length. This allows the tendon to elongate during stressing.

The corrugated plastic sheathing acts as a protective membrane preventing intrusion of any corrosive substances. The cement grout around the threadbar provides corrosion protection by embedding the bar in an alkaline environment. The threadbar deformations minimize the width and control the distribution of any cracks that develop in the free stressing length, fully maintaining the protective action of the grout cover.

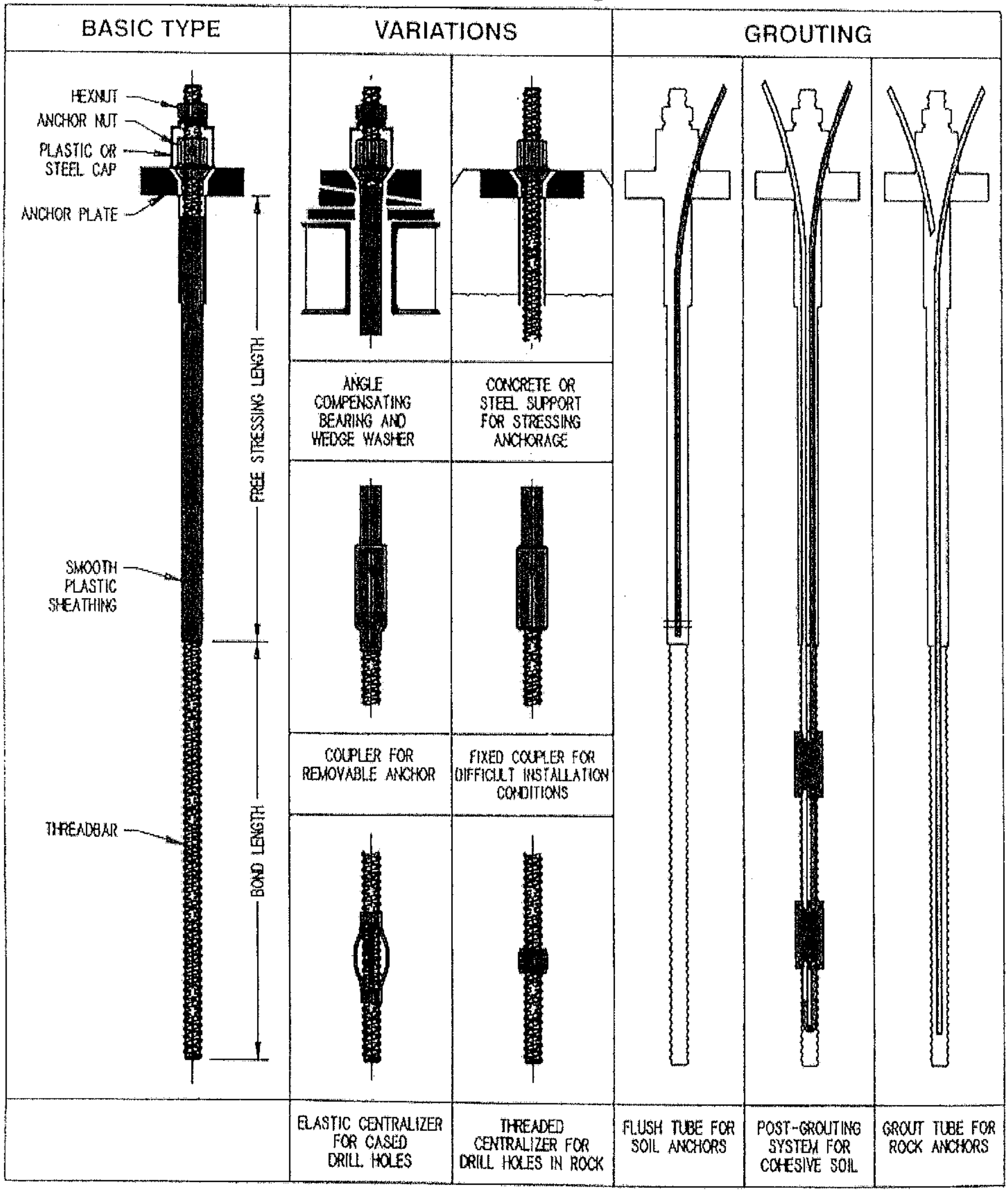
A protective plastic or steel cap filled with a corrosion preventative compound is installed over the anchor nut after stressing, completing the full encapsulation of the anchor tendon. The cap is removable for checking and/or adjusting the force level in the anchor tendon at any time in the future.

Some important notes about the safe handling of high strength steel for prestressing.

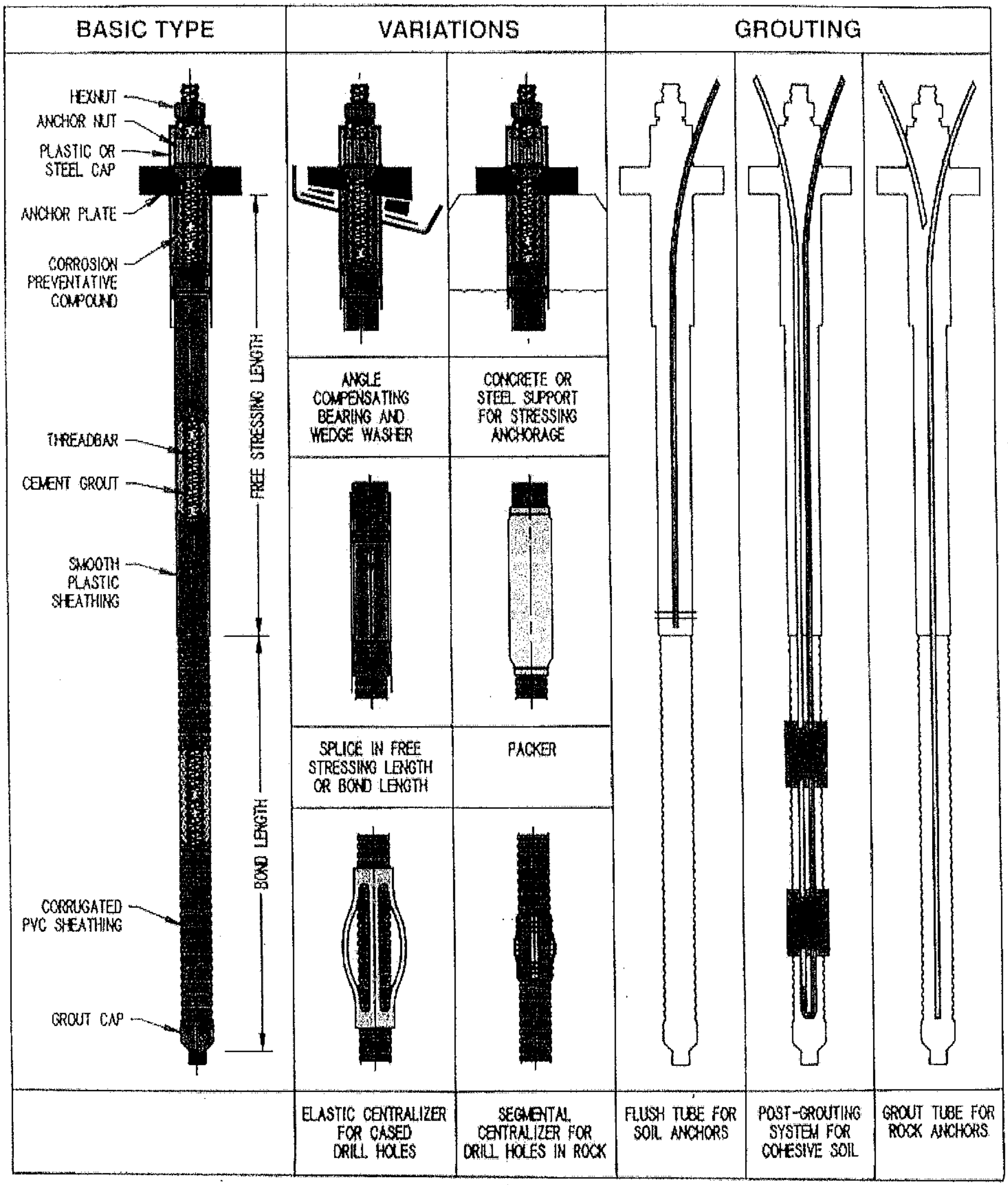
1. Do not damage surface of steel.
2. Do not weld or burn so that sparks or hot slag will touch any particle of steel which will be under stress.
3. Do not use any part of steel as a ground connection for welding.
4. Do not use steel that has been kinked or contains a sharp bend.

Disregarding these instructions may cause failure of steel during stressing.

DYWIDAG Threadbar Anchors with Single Corrosion Protection



DYWIDAG Threadbar Anchors with Double Corrosion Protection



DYWIDAG Bar Rock and Soil Anchors

Prestressing Steel Properties – ASTM A722

Anchor Size		Ultimate Stress f_{pu}		Cross Section Area		Ultimate Strength (f_{pu} A_{ps})		Prestressing Force						Nominal Weight (bar only)		Maximum Bar Diameter	
								0.80 f_{pu} A_{ps}		0.70 f_{pu} A_{ps}		0.60 f_{pu} A_{ps}					
in	mm	ksi	MPa	in²	mm²	kips	kN	kips	kN	kips	kN	kips	kN	plf	kg/m	in	mm
1	26	150	1030	0.85	548	127.5	567	102.0	454	89.3	397	76.5	340	3.01	4.48	1.20	30.5
		160*	1100	0.85	548	136.0	605	108.8	485	95.2	423	81.6	363	3.01	4.48	1.20	30.5
1¼	32	150	1030	1.25	806	187.5	834	150.0	662	131.3	584	112.5	500	4.39	6.54	1.46	37.1
		160*	1100	1.25	806	200.0	890	160.0	707	140.0	623	120.0	534	4.39	6.54	1.46	37.1
1⅜	36	150	1030	1.58	1018	237.0	1,055	189.6	839	165.9	738	142.2	633	5.56	8.28	1.63	41.4
		160*	1100	1.58	1018	252.8	1,125	202.3	899	177.0	787	151.7	675	5.56	8.28	1.63	41.4
1¾	46	150	1030	2.62	1690	400	1,779	320	1423	280	1245	240	1068	9.23	13.74	2.00	51.0

*Available on special order.

Steel Stress Levels

Dywidag Bars may be stressed to the allowable limits of ACI 318. The maximum jacking stress (temporary) may not exceed 0.80 f_{pu}, and the transfer stress (lock-off) may not exceed 0.70 f_{pu}.

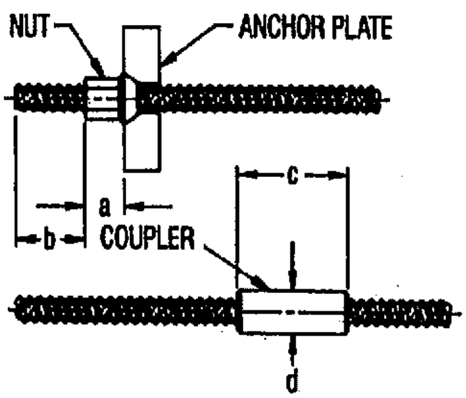
The final effective (working) prestress level depends on the specific application, installation procedure, stressing sequence and the rigidity of the structural system. In the absence of a detailed analysis of the structural system, 0.60 f_{pu} may be used as an approximation of the effective (working) prestress level.

Dywidag Bars may be used individually or in multiples depending upon the magnitude of force requirements or upon drilling considerations.

Actual loss calculations require structural design information not normally present on contract documents.

Hardware Dimensions

Bar		in	mm	in	mm	in	mm	in	mm
Diameter		1	26	1¼	32	1⅜	36	1¾	46
Anchor Plate Size		5 x 5 x 1.25 4 x 6.5 x 1.25	130 x 130 x 32 100 x 165 x 32	6 x 7 x 1.50 5 x 8 x 1.5	160 x 160 x 38 130 x 200 x 38	7 x 7.5 x 1.75 5 x 9.5 x 1.75	180 x 190 x 25.4 130 x 240 x 45	9 x 9 x 2.25 -	230 x 230 x 57.2 -
Nut Extension	a	1.875	50.0	2.5	63.5	2.75	70	2.875	74
Min. Bar Projection	b	3	76.2	3.5	88.9	4.00	100	3.625	92
Coupler Length	c	5.5	140	6.75	170	8.625	220	6.75	173
Coupler Diameter	d	2	50.0	2.375	60.325	2.625	67	3.125	79



Minimum Anchor Diameter

Nominal Bar Diameter		Corrosion Protection											
		Without				Single				Double			
		Without Coupler		With Coupler		Without Coupler		With Coupler		Without Coupler		With Coupler	
in	mm	in	mm	in	mm	in	mm	in	mm	in	mm	in	mm
1	26	1.20	30.5	2.000	50.00	1.625	41.28	2.125	53.98	2.375	60.33	2.500	63.50
1¼	32	1.46	37.1	2.375	60.00	1.875	47.63	2.500	63.50	2.875	73.03	3.125	79.38
1⅜	36	1.63	41.4	2.750	67.00	2.000	50.80	2.875	73.03	2.875	73.03	3.125	79.38
1¾	46	2	50.8	3.125	79.38	2.5	63.5	3.25	82.55	3.5	88.9	4.125	105

DSI

DYWIDAG Anchor Design

The spacing, inclination, length and the load applied of each anchor depend on the local soil or rock conditions. The available drilling equipment and the structural capacity of the other support elements, such as wales, lagging or a concrete retaining wall, may dictate the capacity and configuration of anchors. A factor of safety of 1.5 to 2.5 should be utilized in anchor design.

For rock anchors, the shear stress on the rock socket perimeter is used to size the bond length. For soil anchors, the bond length is generally assumed on the basis of experience and site testing. Field testing should always be conducted to verify design assumptions.

Pull out tests verify that the bond capacity of the threadbar in grout exceeds the recommendations of ACI 318. The threadbar grout interface does not control the bond length. Bond in cohesive soils can be considerably increased using the Dywidag Postgrouting System.

The stressing length depends on the assumed failure plane and/or the size of the rock or soil mass necessary to resist the anchor force. A minimum stressing length of 15 ft. is recommended, so that small movements in the retaining system will not result in a major loss of prestress force.

Dywidag Anchor Installation

Selection of the drilling method depends on the number of anchors, the composition of the soil or rock, availability of equipment and the required diameter of the hole. The selection of the tools and techniques should be left to the discretion of the drilling contractor where practical. The depth of the bore hole should be based on site tests.

The diameter of the bore hole should exceed the maximum diameter of the anchor by at least 1". If centering devices are used, larger holes are required.

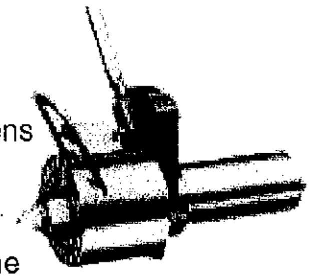
Grouting

For rock anchors, bore holes should be pressure tested to determine water leakage before the anchors are installed. Consolidation grouting, redrilling and retesting are required where water seepage is excessive.

After the anchor is installed in the bore hole, the bond length is grouted. Rock anchors and anchors in cohesive soils are generally grouted without pressure. Soil anchors in loose granular material are pressure grouted while the drill casing or auger is withdrawn.

Dywidag Postgrouting may be used for the installation of anchors in cohesive soils and non-cohesive soils. This technique permits additional grouting operations after the primary grout has cured. Using a series of valves in a preplaced grout pipe, grout can repeatedly be injected under high pressure. Regrouting displaces the previously injected grout and increases the anchor capacity.

Stressing



In stressing, an electrically powered hydraulic jack with built-in socket wrench tightens the anchor nut. The jack fits over a pull rod designed to thread onto the threadbar extension protruding from the anchor nut. Elongation of the anchor can be measured directly or can be monitored by a counter on the jack. Hydraulic pressure is measured by a gauge on the hydraulic pump. Discrepancies of more than 10% between elongation and gauge reading should be investigated. Lift off readings should be taken to determine the applied prestress force. Movement of the structural system must be considered.

Testing

Prior to the installation of any production anchors, test anchors should be installed to verify all design assumptions, including anchor length. Test anchors should be proof stressed to 80% of the guaranteed ultimate strength of the Dywidag Threadbar. After 24 or more hours, readings may be required on selected anchors to determine creep behavior.

All production anchors should also be proof stressed but the load need not be held for an extended period.

DYWIDAG Multistrand Anchors

DSI's Multistrand Rock and Soil Anchor System is based on the proven prestressing technology of the Dywidag Post-Tensioning System and decades of experience in anchor technology. The system is extremely versatile and can be adapted to meet almost any project requirement.

Large Forces

Although there is no theoretical limit to the capacity of a multistrand anchor, practical considerations such as drill hole size and the availability of material handling equipment limit the size of an anchor to 61-0.6″ (15.2 mm) dia. strands. Larger anchors can be manufactured but the practicality and economics of their use should be thoroughly evaluated before they are incorporated into a design. Very large anchors should be avoided in order to assure a satisfactory force redistribution in case of an anchor failure.

Long Lengths

No theoretical length limit exists, however, practical drilling and material handling considerations must be considered. For shop fabrication, the practical limit is dictated by total anchor weight.

Small Bending Radius

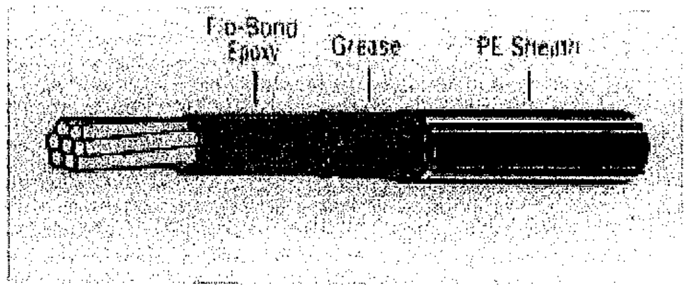
Strand anchors can easily be coiled to fit on a flat bed truck and are well suited for installation where work space is limited.

Corrosion Protection Options

A wide variety of corrosion protection options are available to choose from, depending upon the expected length of service and the aggressiveness of the environment.

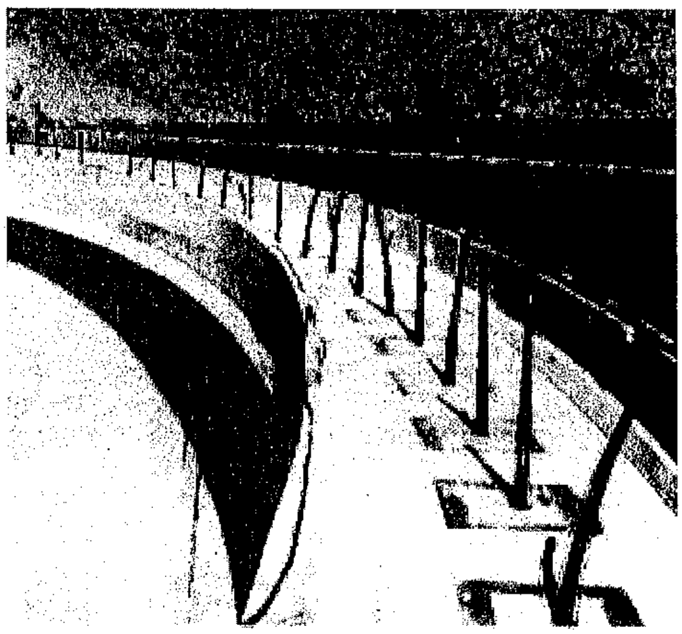
Single Corrosion Protection (SCP) (Type A)

SCP Anchors are used for temporary applications and sometimes for permanent applications in non-aggressive environments. In the bond length, cement grout covers the bare strand. The protection in the free stressing length depends upon whether single stage or two stage grouting is used. For single stage grouting, the free stressing length of each strand is coated with a layer of corrosion preventative grease over which is extruded a tough seamless layer of polyethylene. Grease never comes in contact with the grout in the free stressing length so the bond strength is not affected. For two stage grouting, the grease and PE coating can be omitted. DSI does not recommend the use of bare strand in the free stressing length where the free stressing length remains ungrouted.



Double Corrosion Protection DCP (Type B)

DCP Anchors are used for permanent applications in aggressive or uncertain environments. The strand bundle in the bond length is grouted into a corrugated PE or PVC duct while the individual strands in the free stressing length are greased and sheathed in polyethylene. Quality control may be enhanced by pregrouting the bond length under factory conditions. Drill hole size and cost are significantly influenced by the clearance required by the outer PE duct.



Stewart Mountain Dam, U.S. Bureau of Reclamation. Permanent anchors consisting of 22 and 28 epoxy coated strands.

Double Corrosion Protection DCP (Type C)

Corrosion protection for the anchor tendon can be improved by extending the outer corrugated PE or PVC duct over the free stressing length. In this case, pregrouting of the anchor inside the plastic duct is not recommended because of difficulties which might be encountered during transportation and placing.

Double Corrosion Protection DCP (Type D)

The ideal protection for strand anchors is one in which the strand is totally and permanently protected from the time of manufacture throughout its life. Such protection is provided by epoxy coating the individual strands both externally and internally. Flo-bond Flo-fil® is a rugged, thermally bonded polymer coating that offers maximum corrosion protection, with a bond strength that exceeds that of bare strand. When two stage grouting is used, no additional corrosion protection is required even in applications where the free stressing length will remain ungrouted for an extended period of time.

The Dywidag wedge anchor for epoxy coated strand bites through the coating into the strand, developing 100% of its nominal ultimate tensile strength. Corrosion protection provided by the epoxy is not compromised by the wedge.

Although the cost of epoxy coated strand is higher than bare strand, the total cost of the installed anchor is reduced by eliminating the outer corrugated plastic duct. This makes it possible to minimize the drill hole size, thereby reducing the cost of drilling and grouting.

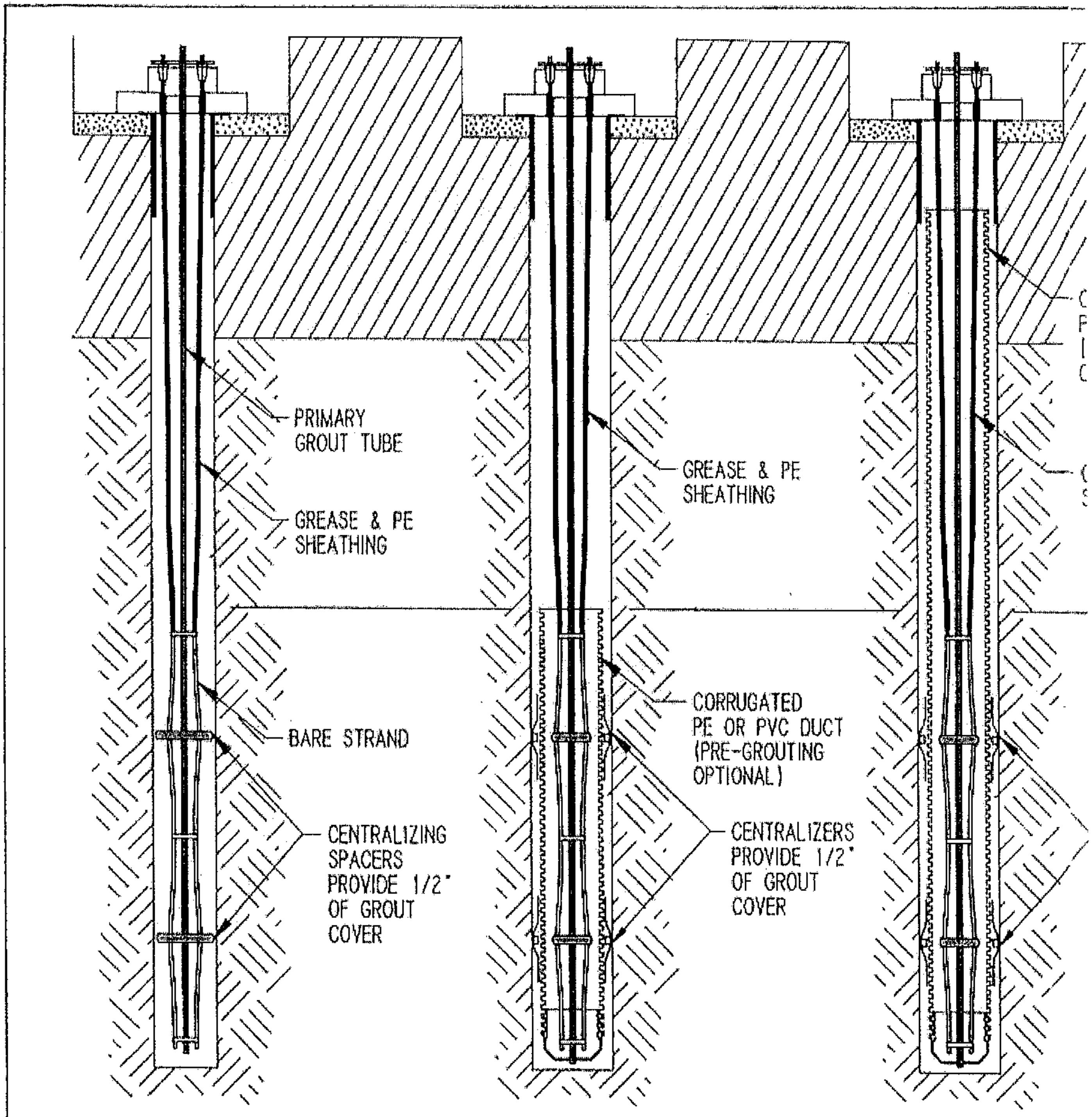
Double Corrosion Protection DCP (Type E)

For anchors in which single stage grouting is desirable, the free stressing length of epoxy coated strand anchors can be coated with a lubricating grease and encased in a seamless extruded PE sheath.

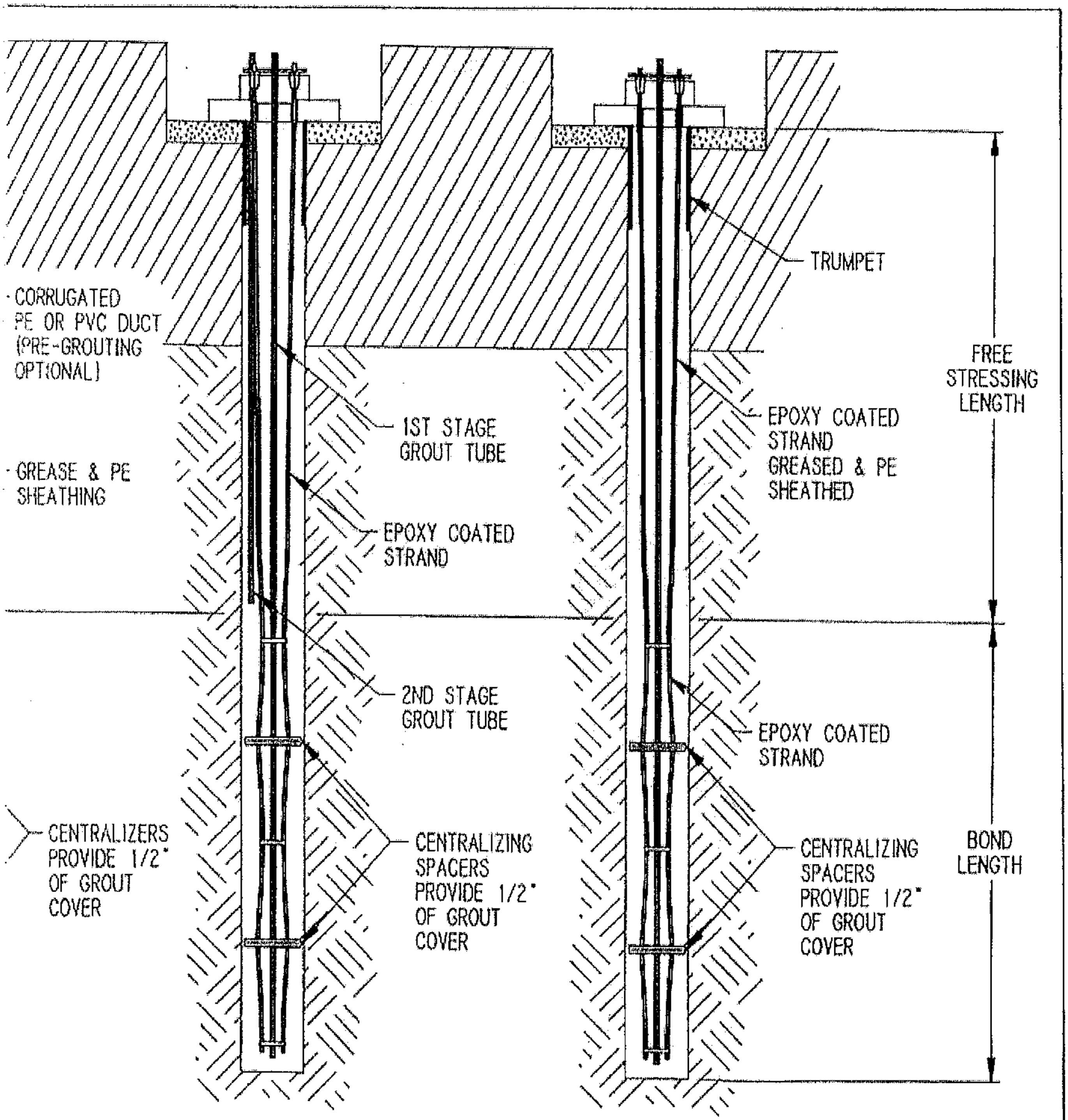
Multistrand Prestressing Steel Properties — ASTM A416

Anchor Size	Nominal Cross Section Area		Nominal Weight (bare strand)		Ultimate Strength (F_{PU} A_{PS})		Prestressing Force					
							0.80 F_{PU} A_{PS}		0.70 F_{PU} A_{PS}		0.60 F_{PU} A_{PS}	
	in²	mm²	plf	kg/m	kips	kN	kips	kN	kips	kN	kips	kN
3 – 0.6	0.65	420	2.20	3.27	175.8	782	140.6	625	123.0	547	105.5	469
4 – 0.6	0.87	560	3.00	4.46	234.4	1,043	187.5	834	164.1	730	140.6	626
5 – 0.6	1.09	700	3.70	5.51	293.0	1,303	234.4	1,043	205.1	912	175.8	782
6 – 0.6	1.30	840	4.40	6.55	351.6	1,564	281.3	1,251	246.1	1,095	211.0	938
7 – 0.6	1.52	980	5.20	7.74	410.2	1,825	328.2	1,460	287.2	1,277	246.2	1,095
8 – 0.6	1.74	1,120	5.90	8.78	468.8	2,085	375.0	1,668	328.1	1,460	281.3	1,251
9 – 0.6	1.95	1,260	6.70	9.97	527.4	2,346	421.9	1,877	369.2	1,642	316.4	1,408
12 – 0.6	2.60	1,680	8.90	13.24	703.2	3,128	562.6	2,503	492.3	2,190	422.0	1,877
15 – 0.6	3.26	2,100	11.10	16.52	879.0	3,910	703.2	3,128	615.3	2,737	527.4	2,346
19 – 0.6	4.12	2,660	14.10	20.98	1,113.4	4,953	890.7	3,962	779.4	3,467	668.0	2,972
27 – 0.6	5.86	3,780	20.00	29.76	1,582.2	7,038	1,265.8	5,631	1,107.6	4,927	949.4	4,223
37 – 0.6	8.03	5.180	27.40	40.78	2,168.2	9,645	1,734.6	7,716	1,517.8	6,751	1,301.0	5.787
48 – 0.6	10.41	6,720	35.50	52.83	2,812.8	12,512	2,250.2	10,009	1,968.9	8,758	1,687.7	7,507
54 – 0.6	11.72	7,560	39.90	59.38	3,164.4	14,076	2,531.5	11,261	2,215.1	9,853	1,898.6	8,446
61 – 0.6	13.24	8,540	45.10	67.12	3,574.6	15,901	2,859.7	12,721	2,502.2	11,131	2,144.8	9,540

DYWIDAG Multistrand Anchors Types (Corrosion Protection Options)



A	B	C
SCP	DCP	DCP
SINGLE STAGE	SINGLE STAGE	SINGLE STAGE

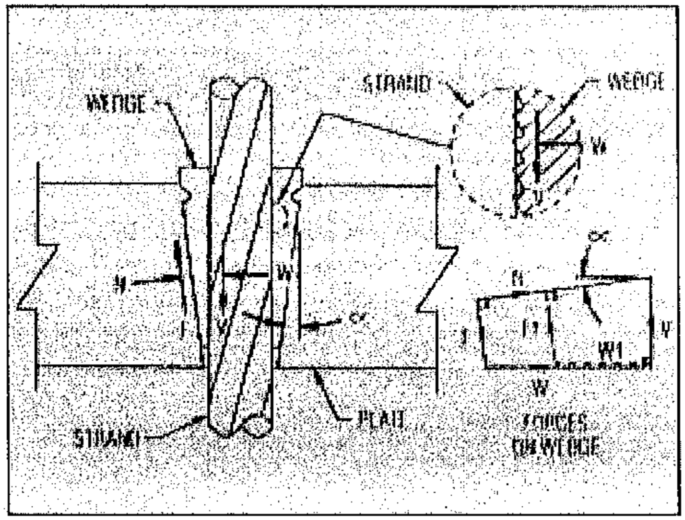


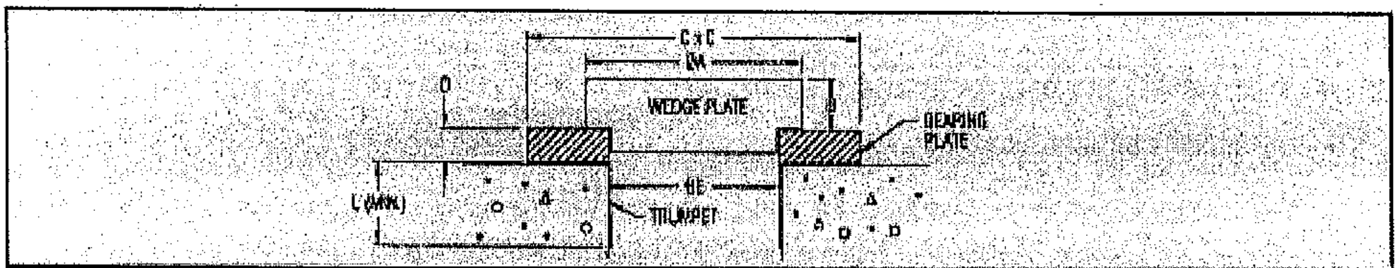
	D	E	ANCHOR TYPE
	DCP	DCP	CORROSION PROTECTION
	TWO STAGE	SINGLE STAGE	GROUTING

Stressing Anchorages

The prestressing force in each strand is maintaind by individual 3-part wedges. The wedge segments grip the strand by means of tooth shaped threads which are forced into the surface of the strand wires as the wedge is drawn Into the wedge hole.

Unless provisions are made to allow the wedge to move further into the wedge hole (reduced friction force F) in response to increases in the strand force V, the wedge teeth will fail in bending and shear resulting in strand slips and anchor failure. For this reason DSI recommends that wedges for strand anchors, in which the free stressing length remains unbonded, be seated at the highest possible force (0.8 fpu). Subsequent adjustment in anchor force should be made by adding or removing shims. Using this technique the wedge teeth will remain securely embedded in the strand. This is particularly important in applications where anchor load is likely to increase with time due to superposition of external loads or seismic activity.





SYSTEMS A, B, & C										
ANCHORAGE SIZE		4 – 0.6″	6 – 0.6″	8 – 0.6″	11 – 0.6″	14 – 0.6″	18 – 0.6″	25 – 0.6″	34 – 0.6″	54 – 0.6″
WEDGE PLATE	ØA	4.50/114.3	5.38/136.7	6.25/158.8	7.00/177.8	8.00/203.2	8.75/222.3	10.50/266.7	11.50/292.1	14.00/355.6
	B	1.80/45.7	2.20/55.9	2.38/60.5	2.70/68.6	3.12/79.2	3.62/91.9	4.52/114.8	4.62/117.3	5.62/142.7
BEARING PLATE	C x C	8.25/209.6	10.00/254.0	12.00/304.8	13.50/342.9	15.50/393.7	18.00/457.2	21.00/533.4	25.00/635.0	30.00/762.0
	D	1.19/30.2	1.38/35.1	1.50/38.1	1.75/44.5	2.00/50.8	2.38/60.5	2.75/69.9	3.50/88.9	4.00/101.6
	ØE	3.30/83.8	4.00/101.6	4.80/121.9	5.40/137.2	6.20/157.5	6.62/168.1	7.75/196.9	8.75/222.3	11.25/285.8
TRUMPET	L (MIN.)	14.0/355.6	16.0/406.4	20.0/508.0	23.0/584.2	26.0/660.4	28.0/711.2	34.0/863.6	40.0/1016.0	44.0/1117.6
SYSTEMS D & E										
ANCHORAGE SIZE		4 – 0.6″		8 – 0.6″	11 – 0.6″	14 – 0.6″		25 – 0.6″	34 – 0.6″	54 – 0.6″
WEDGE PLATE	ØA	5.00/127.0		7.00/177.8	8.00/203.2	9.00/228.6		12.00/304.8	13.00/330.2	16.00/406.4
	B	2.38/60.5		2.38/60.5	2.75/69.9	3.12/79.2		4.52/114.8	4.62/117.3	5.62/142.7
BEARING PLATE	C x C	8.50/215.9		12.50/317.5	14.00/355.6	16.00/406.4		22.00/448.8	25.00/635.0	30.00/762.0
	D	1.25/31.8		1.50/38.1	1.75/44.5	1.88/47.8		2.75/69.9	3.50/88.9	4.00/101.6
	ØE	4.00/101.6		5.19/131.8	6.00/152.4	6.75/171.5		10.12/257.0	10.25/260.4	13.00/330.2
TRUMPET	L (MIN.)	17.0/431.8		23.0/584.2	27.0/685.8	29.0/736.6		37.0/939.8	43.0/1092.2	47.0/1193.8

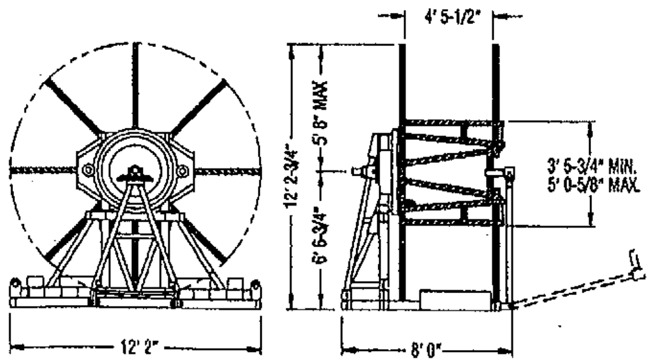
NOTE: Bearing plate design based on A36 steel loaded to 95% of guts

DIMENSIONS: Inch/mm

Uncoiler

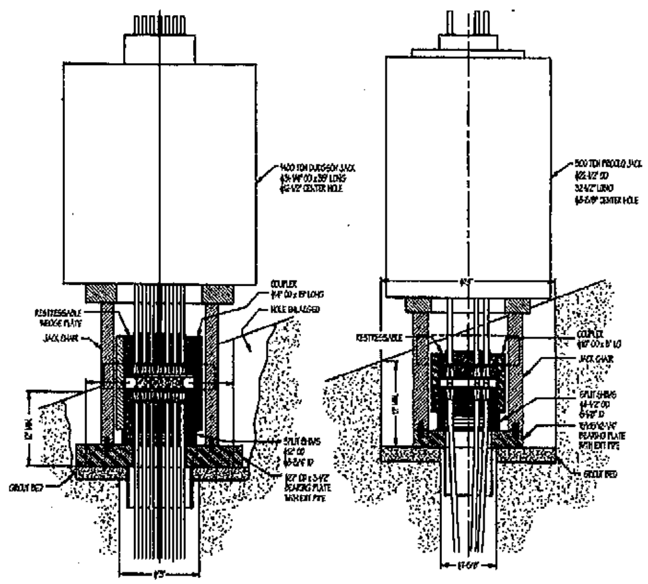
For projects where anchor placement by overhead crane is impractical, DSI can provide a hydraulic powered uncoiler. A unique feature of the Dywidag Uncoiler is the adjustable hub which simplifies the process of placing the anchor in the uncoiler. If necessary, the uncoiler can be used to remove the anchor from the drill hole. Use of the uncoiler, both in installation and/or removal, will reduce the risk of damage to the tendon.

DSI 7.5 Ton Uncoiler



NOTE: REQUIRES AN R35.3 OR EQUAL PUMP TO DRIVE THE UNIT.

Restressable Systems



Threaded Coupler for:
37-0.6" Strand Anchor **14-0.6" Strand Anchor**

Stressing

For installation and stressing efficiency, most DYWIDAG jacks for multi-strand anchors are equipped with internal strand guide tubes with automatic strand gripping and releasing devices. These features make jack installation a fast, one-step operation with small wedge seating loss.

For safety, all jacks feature a check valve which holds the pressure in the unlikely event of hydraulic failure. For reliability, the jacks are equipped with special devices for power seating all wedges simultaneously. Jacks also allow bleed-back to achieve full utilization of the maximum allowable stresses in the anchor.

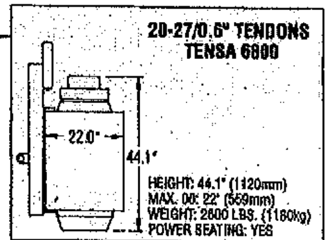
A hydraulic connection and a pressure gauge are provided for all tensioning jacks.

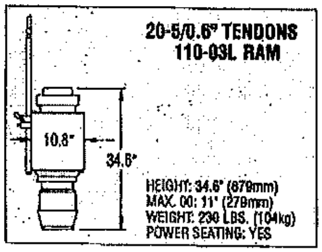
The hydraulic pumps used in conjunction with the jacks can be operated by remote control.

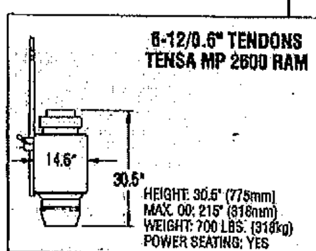
Jack chairs are provided where wedge plate lift off during anchor testing is antisipated.

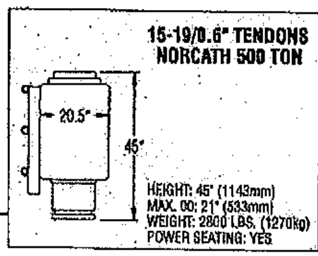
Rams for Anchor Stressing

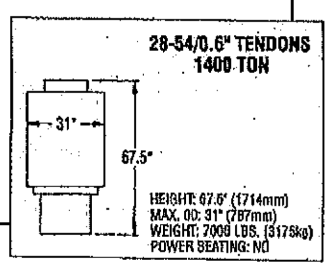
NOTE: Detailed operating and safety instructions are provided with all stressing units. Read and understand these instructions before operating equipment.











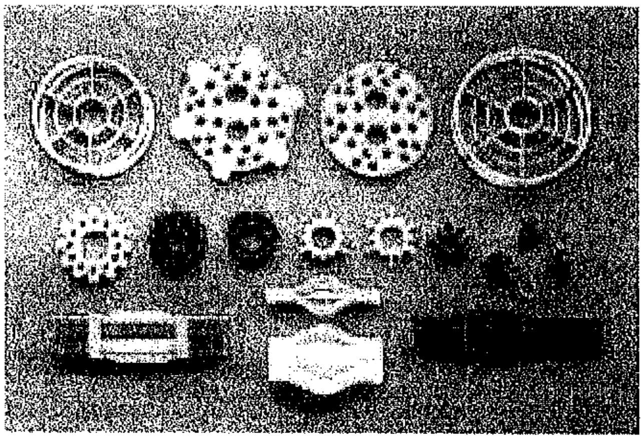
Spacers

The purpose of a spacer is to help insure that grout surrounds each strand for corrosion protection and for bond strength development. Designers should specify the desired distance between spacers (typically 7′ – 10′).

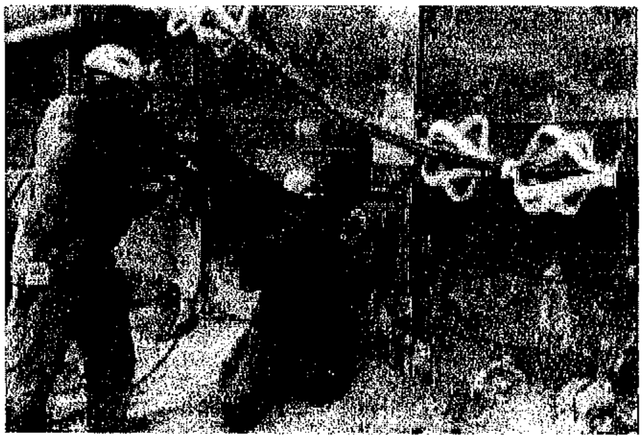
Centralizers

Centralizers are placed over the assembled strand bundle in order to maintain the required spacing between the anchor and the bore hole so that an adequate thickness of grout (minimum 0.5″) surrounds the anchor. A wide variety of centralizers are available depending upon the anchor type

Occoquan Dam, Fairfax County Water Authority. Permanent tie down anchors 40-54 epoxy coated strands.



Typical spacers and centralizers.



Los Angeles Public Library. Permanent tie backs using epoxy coated strand.

DYWIDAG-SYSTEMS INTERNATIONAL, USA INC.

Corporate Headquarters
North Central Division
320 Marmon Drive
Bolingbrook, IL 60440
Tel (630) 739-1100
Fax (630) 972-5517

North East Division
15 Industrial Road
Fairfield, NJ 07004
Tel (973) 276-9222
Fax (973) 276-9292

South East Division
4732 Stone Drive
Tucker, GA 30084
Tel (770) 491-3790
Fax (770) 938-1219

South Central Division
1801 North Peyco Drive
Arlington, TX 76001
Tel (817) 465-3333
Fax (817) 465-3969

Western Division
2154 South Street
Long Beach, CA 90805
Tel (562) 531-6161
Fax (562) 531-2667

Latin America Division
15 Industrial Road
Fairfield, NJ 07004
Tel (973) 276-9222
Fax (973) 276-0249

Visit Our Website:
www.dywidag-systems.com

E-mail:
dsiamerica@dsiamerica.com

DYWIDAG-SYSTEMS INTERNATIONAL, CANADA LTD.

Eastern Division
65 Bowes Road, Unit #5
Concord, ON L4K 1H5
Tel (905) 669-4952
Fax (905) 669-2148

Western Division
19433 96th Avenue, Ste. 103
Surrey, BC V4N 4C4
Tel (604) 888-8818
Fax (604) 888-5008

DYWIDAG-SYSTEMS INTERNATIONAL, ASIA-PACIFIC

Corporate Headquarters
25 Pacific Highway
Bennetts Green, NSW 2290
Australia
Tel +612 4948 9099
Fax +612 4948 4087

12/00 2M